HYDRAULIC MANUAL.

PART I.

CONSISTING OF

WORKING TABLES

AND

EXPLANATORY TEXT,

INTENDED AS A

GUIDE IN HYDRAULIC CALCULATIONS

AND

FIELD OPERATIONS.

BY

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PREFACE TO THE THIRD EDITION.

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presenting this third edition to the public, it is unfortunately my ty to apologise to those interested in the work for the delay that s taken place in its publication; this, however, has been due to cumstances over which I discovered eventually that I had but little ntrol. To avoid disappointing the public generally, and prevent em from expecting to find anything in this book that is not in it, it necessary to state the intentions and scope of the work.

The object of this Manual is to aid the hydraulic engineer in his lculations by means of a collection of working tables based on the ost improved modern principles, and by a small amount of text tting forth these principles and giving all the necessary formulæ in concise manner; also to serve as a guide in hydraulic field operaons by giving short résumés of the modes adopted in the field by e engineers whose experiments have been particularly eminent in roducing practical and theoretical results.

A few miscellaneous paragraphs on various hydraulic subjects are so attached with the hope that some of them may prove of interest, d that others may show the state to which the collective experience the past has arrived, unsatisfactory though it may be in many stances.

In such a work, which is necessarily a compilation, the principal ject has been to avoid as much as possible any attempt at originality, hich might defeat the object of the Manual, and at the same time to corporate the most recent information in the form most convenient r practical application, while not neglecting any of the more ancient t still useful modes and formulæ of calculation that have not yet en superseded.

The works principally consulted, and from which extracts and information have been taken, are—D'Aubuisson's "Hydraulics," D'Arcy and Bazin's "Recherches Hydrauliques;" the "Cultur-Ingénieur," for 1869 and 1870, containing the valuable articles of W. R. Kutter, of Bern; Claudel's Tables, constructed on the system of Dupuit; the Mississippi Report of Captains Humphreys and Abbot; the Lowell experiments by Francis; the "Hydraulics of Great Rivers," by J. T. Révy; also, in a small degree, Box's "Hydraulics," Neville's well-known work on the same subject, Stoddard's and Dwyer's works, Spon's "Dictionary of Engineering," Hurst's Manual, some ancient numbers of various periodicals and cyclopædias, and some articles in the Roorkee professional papers, by Colonel Dickens, Mr. Burge, and Mr. J. H. E. Hart.

In addition, my thanks are especially due to the latter gentleman, for placing at my disposal his valuable MSS. on dams and walls, and to a friend for his on towage.

The Second Part of the Manual, annexed to the first in accordance with the wishes of the Secretary of State for India, consists entirely of hydraulic and meteorological statistics, the former principally, and the latter altogether, Indian.

The hydraulic statistics may be useful for reference in connection with works of irrigation, storage, and river-improvement in any part of the world, but more especially in hot climates. It has not, however, been found advisable to incorporate with them any statistics of irrigation suitable to England or to cold climates generally, because, though the irrigationists in England have certainly achieved an important success in demonstrating unmistakeably that theirs is the only practical mode of dealing with sewage, and are likely to carry out such matters on a more extended scale; yet, in the first place, this system of sewage irrigation differs greatly from the more simple watering practised in warm countries; and, in the second place, the experiments and results obtained at Croydon, Barking, Merthyr Tydvil, Aldershot, and the few other places, do not appear to admit of satisfactory comparison, or to afford a guidance useful under other local circumstances and conditions, either as regards amount or intermittency of supply.

Such of the hydraulic statistics as relate to India were mostly col-cted by myself personally, in the various provinces of India from the ifferent local officials and Government records, and reduced to their resent shape. These cannot be expected to be of so much interest to ngineers of exclusively home practice as to those of more extended ex-erience; and again, the results shown by them may appear, in the eyes of many, to be small in comparison with what might have been done in India under a more favourable administration. While the latter is doubtless true, its counterpart is no less so; it is also surprising that so much has been done under such extreme administrative and financial difficulties; in fact, there is every reason to believe that, had it not been for the energy and great administrative abilities of the former Inspector General of Irrigation, General Strachey, all irrigation works in India would probably have remained at a standstill from 1869 till now, and perhaps longer.

At present, the older canals are being rectified, and new works gradually carried out. The results are not entirely satisfactory in all cases, nor is it possible that they should be; they are, however, on the whole, extensive results, showing an actual and a progressive development of irrigation not existing in any other country, which have not hitherto been collected and impartially set forth in a form conveniently for reference. In the present edition, some modern additions, relating to the years from 1870 to 1873, have been made from India Office records, kindly placed at the disposal of the author by the Under Secretary of State for India. Such statistics as relate to England, France, Italy, and Spain have, in every case, the source from which they were taken mentioned with them.

In all of them, whether tabular or in the form of brief accounts, the object has always been expressly to avoid introducing anything simply because it might be of interest, and to limit myself to simple facts and achieved results that may be useful to engineers for reference. In one or two cases rather doubtful statistics have been introduced, to which foot-notes are attached: this, however, was unavoidable under the circumstances of the case, which were particularly difficult, the voluminous records of India, both at home and abroad, being generally destitute of anything approaching to a catalogue raisonnée, although filed and indexed, according to certain principles, with extreme care. The difficulties, then, had to be overcome in the first place by wading through an immense quantity of matter in order to obtain but a few facts, and in the second place by availing myself of the kind aid of several officials, which materially shortened that labour: to these, therefore, and more especially to the present Secretary in the Geographical Department of the India Office, and to Dr. Macnamara, of Calcutta, I beg to offer my best thanks.

The Indian meteorological statistics here given were also, with the exception of those from 1871 to 1873, collected in India by myself, being supplied by the various meteorological reporters, and afterwards reduced and worked into the present form as most suitable for reference for engineering purposes. They are the first general collection yet made, and include rainfall statistics of all India, and other meteorological statistics of use to the engineer. For the principal portion of them I am indebted to Mr. Blanford of Calcutta, Mr. Chambers of Bombay, and Dr. Murray Thompson of the Panjab: those for the Madras Presidency, excepting the older rainfall data, are unfortunately less complete. The remarks on the meteorology of India, drawn up by myself, are offered as a general account and explanation of the meteorological conditions of India as far as they are at present known.

With regard to the alterations effected in this edition, they principally consist of replacing two or three of the former working tables by new ones, and adding such new tables as the modern system of Kutter absolutely requires: the appendix of miscellaneous tables and data, which are taken from various works and other sources, is slightly enlarged; the text is generally re-written or re-arranged, some additions being made to the article on modules, including a description of a new module of the author's. The hydraulic statistics, as well as the Indian meteorological statistics, have been increased by all such matter as has reference to data available only since the author's departure from India in 1872; the sole matter expunged being the description of the author's evaporameter.

L. D'A. J.

ROYAL INSTITUTION, ALBEMARLE STREET,
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PART I.

TEXT.

CHAPTER I.—EXPLANATION OF THE PRINCIPLES AND FORMULÆ ADOPTED IN CALCULATION AND APPLIED IN THE WORKING TABLES.

CHAPTER II.—ON FIELD OPERATIONS AND GAUGING; WITH BRIEF ACCOUNTS OF THE METHODS ADOPTED BY VARIOUS HYDRAULICIANS.

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WORKING TABLES.

MISCELLANEOUS TABLES.

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PART I.

CHAPTER I.

EXPLANATION OF THE PRINCIPLES AND FORMULÆ ADOPTED IN CALCULATION AND APPLIED IN THE WORKING TABLES.

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1. Hydrodynamic Theories. 2. Notation and Symbols. 3. Rainfall, Supply, and Flood Discharge. 4. Storage. 5. Discharges of Open Channels and Pipes. 6. Section of Channels and Pipes. 7. Other Theories of Flow. 8. Velocities in Section. 9. Bends and Obstructions. 10. Discharges of Sluices and Weirs. 11. Discharge from Basins, Locks, and Reservoirs. 12. Application of the Working Tables.

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1. HYDRODYNAMIC THEORIES.

THE science of hydraulics, yet in its infancy, may be said to depend, as far as its practical application by the hydraulic engineer is concerned, on a combination of certain known laws with the empirical results of observation and experiment; the former few in number, and eliminated principally by the philosophers and mathematicians of the past; the latter also few, and, if we except the old observations which were carried out on a very petty and limited scale, exceedingly modern. Previous to the experiments of d'Arcy in 1856, little was known about the velocities and discharges through pipes; until the operations of Captains Humphreys and Abbot on the Mississippi in 1858, the discharge of large rivers was a comparatively unexplored subject; in 1865 the experiments of Bazin led the way to a more accurate knowledge of the discharges and velocities of open channels. Before this time the less important subjects alone had been investigated to any practical purpose, such as the vena contracta, the discharges through small orifices, over certain forms of overfall, and through short and small pipes, the discharges from reservoirs, and the velocities in troughs 18 inches wide. There was, however, plenty of theory, and a large number of formulæ, some of them exceedingly complicated in form, mostly resulting from a number of superimposed theories, the more ancient of which were based on very limited experiments: in fact, the mode often adopted seems to have been to assume a new form of formula, and to prove it by a few partial experiments, a principle worthy of ancient soothsayers, and which, had it been further supported by traditionary and name-reverencing hydraulic schools of believers, could only have resulted in prolonged and permanent error. At present even, a reference to some works comparatively recently published in England will show formulæ to be supported by a most heterogeneous collection of experimental data; discharges of pipes irrespective of their material or internal surface, of large and small rivers irrespective of the quality of their beds and the bends in their courses, of canals in any material, down to wooden troughs, all seem to prove the correctness of a fixed formula having an unvarying constant coefficient: other works again having greater accuracy of result in view go to the opposite extreme in method, and recommend the adoption of two distinct formulæ for cases in which the principle involved does not even seem to vary in the least, as for instance, in discharges through pipes with low velocities, a formula distinct from that for those with high velocities is often adopted; this, amounting to a method of successive approximation imperfectly worked out, is almost as unfortunate as the other. From a continuance of this, however, the modern experiments have already saved us to a great extent, and further and more extended experiment will probably relieve us from it altogether.

At present, therefore, the hydraulic engineer is more dependent for correctness of calculated result on the so-called empirical data obtained by experiment, and put into convenient form, than on any purely mathematical theories or laws. The correct application of all known mechanical laws cannot, however, fail to be valuable in cases admitting of them; those relating purely to hydrodynamics are comparatively few, and the most important and best known of them are the three following:—

First. If fluid run through any tube of variable section kept constantly full, the velocities at the different sections will be inversely as the areas, or

$$A V = A' V'.$$

This theory of uniformity of motion is in practice supposed to hold good with reference to mean velocities of discharge; which is actually little more than assuming a theoretical velocity that will fulfil the conditions of the law, in order to render calculation convenient. There is no reason to believe that actual velocities in a tube of variable section would all vary inversely with the area of cross section; hence this theory is not one that throws any light on the laws of absolute velocity.

Second. The velocity of a fluid issuing from an orifice in the bottom of a vessel kept constantly full, is equal to that which a heavy body would acquire in falling through a space equal to the depth of the orifice below the surface of the fluid, which is called the head on the orifice; or by way of formula

$$V = (2g\ H)^{\frac{1}{2}}$$

where H = the head, and g = force of gravity. The quantity g represents the accelerating force of gravity, which varies at different places on the earth's surface and elevations above the mean sea level, and is also affected by the spherical eccentricity of the earth at the place, a quantity that again varies with the latitude; above the earth's surface g varies inversely with the square of the distance from the earth's centre, below the earth's surface direct with the distance from the earth's centre; to obtain the exact value of g, d'Aubuisson's formulæ applied to English feet are—

$$r = 20\,887\,540\,(1 + \cdot001\,64 \cos 2\,l)$$

$$g = 32\cdot1695\,(1 + \cdot002\,84 \cos 2\,l)\left(1 - \frac{2e}{r}\right).$$

The values of this formula for different latitudes and elevations are given in Working Table No. I., and the values of g, obtained from observation at different latitudes, are given in Table No. I. of the Hydraulic Statistics. For purposes of ordinary calculation in England, and hence throughout these tables, g is generally taken as 32·2 feet per second; in India, however, it would be more correct to use 32·1; but the convenience of using English data will probably outweigh that of this exactness until the science of hydraulics can produce far more accurate results than now.

The above theory supposes that the orifice is indefinitely small, neglects the conditions and size of its sectional area, friction, the pressure of the atmosphere, and the resistance of the air to motion, which increases with the square of the velocity of the issuing fluid; the practical application that shows its discrepancies most strongly is the fact that the height of a jet is never equal to the head of pressure on it.

Third. The theory of flow, which is a combination of the two previous theories in a modified form, assuming

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both uniform motion and the principle of gravitation, and is best expressed in the form of a formula—

$$V = (fg\,RS)^{\frac{1}{2}}$$

where V = the mean velocity generated.
R = mean hydraulic radius of the water section.
S = the sine of the slope of the water surface.

This formula is a simple equation of the accelerating force of gravity down an incline with the retarding force of friction at any section at right angles to the course of flow, namely:—

$$gS = \left(\frac{V^2}{Rf}\right)$$

since, for uniform motion, the total accelerating force is equal to the total resistance.

This theory is the basis of calculation of flow in full tubes, and in open channels and unfilled pipes, where the principle still holds, though the equation should be strictly modified, the air above giving a resistance as well as the surface of the channel or tube below, though in a less degree.

However rigid these theories may appear in neglecting important points, they are yet generally true in the abstract, and no substitutes for them have yet been discovered; the consequence is that all hydraulic calculations are made to depend on them, their defects being made up by applying to them experimental coefficients, in preference to endeavouring to obtain theoretical accuracy by introducing into them niceties of theory that might fail in obtaining trustworthy results. It becomes, therefore, one of the important duties of a hydraulic engineer to apply these partly empirical formulæ with care and circumspection, especially guarding against taking for granted the formulæ and tabular results of different calculators, which vary in form and in result to a very great extent; some authors even giving one-third more discharge than others as due to the same data. During practical work, again, time forbids a lengthy examination of principles; for this reason, therefore, this short chapter is given as an easy guide to the proper management and application to every-day wants of the working tables attached, which are based on the most improved modern principles.

2. NOTATION AND SYMBOLS.

To ensure clearness and rapidity of application of these theories, it is absolutely necessary that the nomenclature should be neither doubtful nor inconvenient, that the symbols be free from confusion, and the units of time, weight, and measurement, once adopted, generally adhered to as much as possible; this alone can cause the form of a formula to give at a glance any definite idea of the values of its terms and expressions.

The English foot has been generally adopted in this work as the unit of length, surface, and capacity, being the measure ordinarily used for heights and depths, as well as distances in survey work, and being now more capable of extended application than either the yard, link, or inch; the second has been generally taken as the unit of time, so that the numbers expressing discharges and velocities, which often are high numbers, may be as small as possible. This has been found to be perfectly manageable in practice. In the canal departments of Northern India the engineers have succeeded in abolishing chains, yards, and inches from their plans, estimates, and calculations, and in adhering generally to the second as a unit of time; they have also, on the Bari Doab Canal, adopted a mile of 5000 feet to the exclusion of the statute mile of

5280. This decimal system of measures, while retaining the use of a familiar unit, is found to save much needless labour in calculation, and at the same time has the great advantage of facilitating the conversion of foreign data and formulæ; the principal difficulties to contend with are the old habits of measuring water supply for towns in gallons instead of cubic feet, and of using dimensions of pipes in inches, instead of tenths of a foot; these obstacles will probably gradually disappear.

As regards the metrical system, although it is now adopted in all the civilized countries in Europe except our own, there seems little hope of its replacing our own measures to the entire exclusion of them for some time yet; hence it would not have been an advantage to have constructed the accompanying working tables on the metrical system, nor to have adopted it throughout this work in the data and formulæ; but as English engineers are now conversant with metrical measures, all such foreign formulæ and data mentioned are generally left in their original form, their conversion not serving any important purpose, but rather, on the contrary, causing complication needlessly.

Whether the decimal metrical system will hold its own for a very long time is yet a matter of considerable doubt: the number ten is not by any means in itself a convenient number for purposes of calculation, it is neither composed of a large number of factors, and hence admitting of easy subdivision, nor are its roots easily obtained; its use involves the necessity of referring to tables of logarithms in the greater part of the calculations made by engineers and scientific men; its sole advantage is a perfectly fortuitous one; it was chosen in ancient times as the first number to be represented by two digits, and the digital advantage it now possesses is perhaps its only one.

Should in the future any new notation come in vogue, which would readily enable the calculator to dispense with half of his logarithmic calculations, the advocates of the decimal system will then be looked upon as antiquated obstructors of progress.

For the present, however, the adoption of a decimal system seems absolutely inevitable, although it seems doubtful whether the English will first adopt one based on units familiar to them, or will change at once to the metrical system in its entirety.

The hydraulic engineer can, however, very conveniently adopt a decimal system based on the English foot for measures; nor apparently are there any very good reasons why the railway engineer should not do so also, except perhaps the tradition-loving habits of the multitude, and the meddlesome legislation in social matters under which we suffer, which enforces on him the adoption in Parliamentary plans of the whole of the old measures.

The advantage of adhering to one set of symbols in hydraulic formulæ, which sometimes appears very complicated, is sufficiently evident; with this view, therefore, the following general notation is drawn up, and the velocity notation of the Mississippi survey also attached for purposes of reference.

General Notation.

N = catchment area drained.
Q = discharge; q = discharge per square mile drained.
V = mean velocity of discharge, V_1, &c., other velocities.
V_x = maximum velocity in the cross section.
A = sectional area; a, a_1, a_2, subsidiary areas.
P = wetted sectional perimeter.
H = mean head or fall; h, h_1, h_2, subsidiary heads.

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R $=$ mean hydraulic radius $= \frac{A}{P}$

R_1 $=$ prime hydraulic radius $= \frac{A}{P + W}$

S $=$ hydraulic slope in terms of its sine $= \frac{H}{L}$;

thus $S = \frac{1}{500} = \cdot 002$ for a slope of 1 in 500.

L $=$ a longitudinal length taken in the direction of flow; l, l_1, l_2, subsidiary lengths.

W $=$ total transverse width, across the direction of flow; w, w_1, w_2, subsidiary transverse widths.

D $=$ depth from surface level; d, d_1, d_2, subsidiary depths.

T $=$ total time of discharge; t, t_1, t_2, subsidiary times.

f $=$ experimental coefficient of fluid friction.

n $=$ experimental coefficient for drainage discharges.

c $=$ experimental coefficient for channel discharges.

m $=$ experimental coefficient for orifice and overfall discharges.

g $=$ velocity acquired by gravity in one second $= 32 \cdot 2$ feet approximately.

All dimensions are generally in feet and decimals, and velocities and discharges are in feet and cubic feet per second.

Velocity Notation of the Mississippi Survey.

v $=$ mean velocity of the river.

V $=$ velocity at any point in any vertical plane parallel to the current.

$V_{100\ 20}$ $=$ velocity at a point 20 feet below the surface at a perpendicular distance of 100 feet from the base line.

U = velocity at any point in the mean of all vertical planes parallel to the current.

U_m = grand mean of the mean velocities in all vertical planes parallel to the current.

U_r = the mean of the bottom velocities in all such planes.

${}_{w_{,}}V$ = velocity at any depth below the surface at a perpendicular distance $w_{,}$ from the base line.

V_o = velocity at the surface in any vertical plane parallel to the current.

V_2 and V_D = velocities at mid-depth and at the bottom in any such plane.

$V_{d_{,}}$ and V_m = the maximum and the mean velocities in any such plane.

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b = sectional constant $= \frac{1{\cdot}69}{(R + 1{\cdot}5)^{\frac{1}{2}}}$.

l = length of a portion of river.

h = difference of level of the water surface at the two ends of l.

$h_{,}$ = the part of h consumed in overcoming longitudinal channel resistances, for a straight, regular course.

$h_{,,}$ = the part of h consumed in overcoming transverse channel resistances or irregularities.

W = river width at any given place.

w = perpendicular distance from the base line to any point of the water surface.

$w_{,}$ = perpendicular distance from the base line to the surface fillet moving with the maximum velocity.

D = total depth of river at any given point of surface.

d = depth of any given point below the surface.

d_s = depth from the surface of the fillet moving with the maximum velocity in the assumed vertical plane parallel to the current.

m = depth from the surface of the fillet moving with a velocity equal to the mean of the velocities of all fillets in the assumed vertical plane parallel to the current.

Δ = maximum or mid-channel depth.

3. RAINFALL, SUPPLY, AND FLOOD-DISCHARGE.

All hydraulic works of irrigation, drainage, storage, water supply, river improvement, and land reclamation, are more or less affected by the amount and periodicity of the rainfall; for many of them careful and trustworthy rainfall statistics and data are absolutely essential; but the nature and amount of detail required vary with the nature of the work; works of storage being those that, perhaps, require the greatest amount of accurate information. In order that these local records should be sufficient to form a correct basis for the engineering data of these latter works, they should comprise observations extending over a period of ten years, or of the local period comprehending a cycle of rainfall from one season of maximum rainfall to another, including years of extreme drought; from these the following results can be deduced:—

1. The mean, maximum, and minimum monthly rainfall, from which the mean and extreme falls for each natural local season, wet, cold, and hot, can be obtained.
2. The mean and maximum daily falls, in twenty-four hours for each month.

3. Special occurrences, hourly falls, longest continuous falls and droughts.

These, arranged in a convenient tabular form, are all the rainfall data that the engineer will generally require.

In most cases, also, and especially in hot climates, evaporation records are also necessary; and sometimes, too, it is advisable to possess other meteorological data, such as those of humidity, temperature, atmospheric pressure, and wind; and, what is often difficult to procure, some data of absorption and percolation that would be applicable to the soils of the district under consideration.

On many of the works before mentioned, the first duty of the engineer is to account for the whole of the downfall, or to discover what becomes of it all, under both ordinary and unusual circumstances, so that he may be able to deal with more certainty of knowledge with that portion of it that more intimately affects his works; as, for instance, the bridge-builder with the floods, the engineer of storage works with the drought, and those of canals and river improvement with both. A geographical and geological knowledge of the catchment area, whose rainfall affects the works, is hence also needful; the boundaries of this area, its lines of watershed and drainage, its disposition as regards prevailing winds, the nature and porosity of its soil, and the amount of vegetation or cultivation on it, as well as any available records from which the quantities of water actually run off by its streams and rivers in various seasons may be arrived at, are all data necessary for establishing satisfactorily a perfect knowledge of the disposal of the whole of the rainfall under any circumstances.

In many instances it is, from want of sufficient informa-on, utterly impossible to obtain this perfect knowledge;

in others, the deficient data may be supplied by approximations known to hold good in other similar cases, and a tolerably correct approximate balance struck between the downfall and the amount evaporated, absorbed, and run off; in any case, however, the engineer may, with time and means at his disposal, gauge the streams and rivers affecting his works, and make correct records of the amount of water run off in them at different seasons of the year, and in exceptional floods. Failing, however, both time and opportunity, such data have to be observed in a rapid manner that will enable him to determine this approximately; such as the section and fall of the rivers, the depths at various stages, floodmarks, and, if possible, a few velocity observations. The results principally required are the flood or maximum discharge, in cubic feet per second, of the river or stream draining the catchment area; its mean discharge throughout the year; and its minimum discharge in seasons of extreme drought, as well as in its ordinary low stage; dividing each of these by the number of square miles of catchment, similar results per square mile are obtained, which, when multiplied by 1·131, express the depth in feet of rainfall run off under each of those conditions. The relation between these quantities and the probable or approximate downpour over the catchment area can then be compared with those known to exist in other similar cases, and a valuable check on these important results thus obtained.

Flood Discharge.—The determination of the quantity of water discharged from a catchment area in a river or stream at a time of extreme flood, is a matter that is very often of the highest importance. Costly bridges have continually been sacrificed, and long lengths of canal damaged for want of sufficient attention having been paid to this subject.

When the data mentioned in the foregoing paragraphs can be obtained, and are properly handled, there is little difficulty in arriving at a correct result; but, as in many cases, only some of these are forthcoming, the bases of calculation are considerably narrowed, and various modes of obtaining a result necessarily varying with the available, have to be adopted.

When the catchment area has to be scaled from a map, and the highest maximum rainfall of 24 hours has to be taken from observations made at perhaps only one or two places near that area, the flood discharge may be approximately obtained by the equation, 6

$$Q = n \times 27 \sqrt{N},$$

where Q = flood discharge in cubic feet per second.
N = catchment area in square miles.
n = a local coefficient chosen with reference to the maximum day's rainfall of the place.

In using this as well as other formulæ of a similar type, records of flood discharges under somewhat similar condition are necessary for reference, in order that a practically correct value of n the coefficient may be assumed. This formula, which was originally adopted in connection with the inconvenient mode of estimating discharges in cubic yards per second or per hour, has very little to recommend it, the values of n being necessarily very wide in range; it still, however, has its adherents.

A more convenient one, having a narrower and more correct range of coefficients, is the following, which is a slight modification of that of Colonel Dickens, having a more extended application. It is

$$Q = n \times 100 \, (N)^{\frac{3}{4}},$$

and its terms are generally similar to those of the last formula. The values of *n* for India, generally lie between 1 and 24: see coefficients at Table XII., Part 2, page lxx. of the Working Tables;—some further values of it applicable to various river basins in India, are also given in the tables of flood discharges at page [8] of the Hydraulic Statistics in the second part of this Manual. The values of the general expression, for a value of $n = 1$, are given for catchment areas of various sizes at pages ix. and x. of the Working Tables, Table IV, Part 1, and the chosen coefficient can be readily applied to these quantities.

The original form of this formula was simply $Q = 825 (N)^{1}$; it was considered applicable only to Bengal and Bahar in the first instance, and afterwards as applicable to all areas in the plains of India that have an annual rainfall of from 24 to 50 inches. It seems, however, more rational to use a variable coefficient depending on a similarity of general conditions, of which the maximum day's downpour is perhaps the most important. In Northern India where this latter is about 1·5 feet in or near hills, and 1·0 foot in the plains, the flood waterway allowed for bridges has generally been based on the assumption that the rainfall run off would amount to 1·0 foot in depth over the whole; and allowance has been made with these data for the flood waterway of the streams and rivers crossing both the Ganges Canal and the Sarhind Canal; in other cases, also, in Northern India, two-thirds of the depth of downpour is assumed to pass off in flood. It is, however, better to use the improved formula given and assume a coefficient suitable to similar conditions of catchment area.

A further attempt at arriving at a flood discharge by means of a formula has been made by Mr. Burge, Resident Engineer of the Madras Railway. His formula given in the Indian Professional Papers is

$$Q = 1300 \frac{N}{L^{\frac{2}{3}}}$$

where Q and N are as before, L = the length of the main river in miles, and 1300 is a coefficient applicable to a maximum downfall of 6 inches in 12 hours, and an area elevated from 500 to 1300 feet above mean sea level, consisting principally of unstratified rocks. It was deduced from observations on 27 bridges, of above 80 feet span, on the Madras Railway, and its results correspond closely with those of recorded flood sections; the errors lying between 4·64 feet too high and 3·40 too low in height of section. He argues most justly that the length of the river necessarily extends the time of the discharge, and hence diminishes the quantity passing off within a certain time; and that also the functions of discharge, the hydraulic slope, the cross section, and the head affected by the sinuosities in greater length, are reduced by it. Admitting this, the same principle would apply not only to the main river, but also to its tributaries; the number and conditions of the tributaries would probably be a more important consideration. Again, there is much difficulty in saying where a main river begins; so much so, that in the first place the introduction of an index of $\frac{2}{3}$ against a coefficient of 1300 would appear to be a needless attempt at exactitude; and in the second place the introduction of the length of the river at all in an equation of this sort is a matter incapable of very extended application; although in the instances from which this formula was laid down it has been very successfully introduced.

A better mode of introducing a function somewhat similar to this would be to apply in the equation the ratio of extreme breadth to extreme length of catchment a ; we have already a formula, the range of whose efficients for India seem to be between 1 and 24—an portant step already gained ; and if this is modified into form,

$$Q = n_{,} \frac{B}{L} 100 (N)^{\frac{3}{4}},$$

ere B = extreme breadth of catchment area,
d L = extreme length of catchment area,
nd $n_{,}$ = a new coefficient,

obtain a more tangible improvement, capable of extended plication. It is unfortunate, however, that for this mula a sufficient number of values of the new cocient are not yet forthcoming ; although in the instances which it has been applied the improvement seems arly manifested in reducing the range, so that for the sent it is, perhaps, generally better to use that from ich it is modified, while in special cases the ratio can easily introduced.

Failing, however, such data as would be needful to ble one to choose a practically correct coefficient for mulæ of this type, it becomes necessary to fall back irely on recorded flood marks, as a means of approxiting to the flood discharge ; and after gauging the charge of the river in its ordinary stage, assume the d discharge to be proportional to it according to the inary formula,

$$\frac{Q}{q} = \frac{A \sqrt{R}}{a \sqrt{r}}$$

ere A is the sectional area up to flood mark, R its draulic mean radius, and a and r are similar quantities responding to the discharge (q) determined by obserion.

4. STORAGE.

Reservoirs generally have for their object either the detention of flood water that might otherwise cause damage, as in works of river improvement, or the utilization of it in canals, of navigation, irrigation, or driving machinery, or for town supply. For the first purpose they must, to effect their purpose, be very extensive, and strongly aided by the natural formation of the country; for the last purpose they are, in one respect, excepting under very favourable conditions, particularly ill-fitted. The collection of drinking-water from the surface of land needs, in the first place, a clean, uncultivated and uninhabited tract of land as a catchment area; and in the second place, the water stored in the reservoir, which is liable to become putrescent, or seriously affected by the organisms, plants, and animalculæ that inhabit stagnant water, requires a very perfect and careful filtration, of a sort beyond the ordinary economic powers of municipalities or public companies. Indeed it is now asserted to be an uncontrovertible fact, that it is to the tainted water of rivers and reservoirs that one-half of most preventible diseases are due, the other half being caused by want of ventilation, faulty drainage, and mistaken modes of managing sewage, or, in other words, that impure air and tainted water are the chief enemies of human life; and there is, therefore, every reason to believe that in the future, when the general public become awake to this, and acquire enough energy to throw of the incubus of vested interests in the form of water companies, both tainted rivers and open reservoirs will be universally condemned as sources of drinking-water supply, and the water filtered, stored, and preserved against impurity by nature in the permeable strata of the earth, will be drawn on in a more scientific and enlightened way than is at present usual, and be considered, as it justly is, a necessary of life. Another quarter of a century may show us scientific men objecting, on sanitary grounds, to the watering of our streets with such water as is now used in our food. It will therefore be only under very favourable conditions, or under circumstances that admit of no better alternative, that the water of storage reservoirs will be used to drink. For extinguishing fires, watering streets, and many other purposes, such water is, however, still valuable under ordinary circumstances.

The determination of the size and dimensions of a storage reservoir is a matter entirely governed by local circumstances and requirements. The assumptions that the area covered by it should bear a certain proportion to that of the catchment area, or that the amount of water stored should be as nearly as possible one-third of the available supply, are not by any means rules to be applied without a very large discretionary power, although there are rules laid down in various forms by different hydraulic engineers that very much resemble these. The object being the collection and retention of a certain amount of water for a definite purpose, and the circumstances being the local formation of the ground and the amount of downpour on the catchment area, all the economic considerations depend on these points.

The intention may either be to store as much water as possible within a certain amount of expenditure of cost, or only a definite amount sufficient for a certain purpose, or to store all that can possibly be obtained with a knowledge that the extreme amount would not be enough. Again, in one case, the quality of the water and the convenience of proximity, or of cleanliness of site, may be considerations outweighing all others. If, therefore, the latter is the case, there are generally not many local conditions answering the purpose within which any choice can be made; and again, if a definite amount be required, the same may be generally said. It is only therefore in the case when the object is to store and utilise as much water as possible that much choice is left to the engineer.

Large artificial reservoirs being generally made on the natural surface of the ground, and bounded in one direction only by an embankment of earth, or a dam of masonry or brickwork, the first object is to choose a site or sites where the greatest amount of water can be stored with the shortest and least amount and length of embankment; for this purpose a river gorge, narrow and precipitous, terminating a great length of country, having a gradual fall towards it, offers the best ordinarily natural conditions; if, in addition, the lateral or transverse slope of the country is also very gradual, it becomes a large natural basin, with one narrow outlet; and if that admits of being easily dammed, an extraordinary advantage not often available presents itself.

The economy of constructing one large reservoir in preference to two or more small ones to hold the same amount would, perhaps, be evident at first sight to most people. The author has, however, met so large a number of persons that believe the contrary, that he is constrained to give the following mathematical proof of it by Graeff.

Let a single reservoir, or rather its contents when full, be supposed to consist of a number of laminæ, or layers of water, the sum of which will equal the total content, and let

K = the height of any one layer;

P and S = the perimeter and surface of its lower side;

P' and S' = the perimeter and surface of its upper side;

en the volume of this layer will be

$$= aK + \frac{bK^2}{2} + \frac{cK^3}{3}\text{ ; where } a = S;$$

$$b = \frac{2P(S' - s)}{K(P' + P)}\text{ ; } c = \frac{(S' - s)(P' - P)}{K^2(P' + P)}\text{ ;}$$

Hence the above expression becomes

$$= \frac{K}{3(P' + P)}\left\{3S\overline{P' + P} + P'\overline{S' - S} + 2P\overline{S' - S}\right\}$$

$$= \frac{K.}{3(P' + P)}\left(P.\,2\overline{S' + S} + P'\,\overline{2S + S'}\right).$$

In the case where the lateral and longitudinal slopes of the ground are uniform, we can imagine the reservoir to consist of one only of these layers; and its content will then represent that of the whole reservoir. In this case the height of the layer will be the extreme depth of water stored, and the quantities S and P will become indefinitely small in comparison with S′ and P′, and may hence be neglected: hence the total volume of water stored $= \frac{KS'}{3}$, and this is the volume of a reversed cone having S′ for its base; a demonstration that proves how rapidly the amount of storage increases with the depth of water, or with the height of the embankment.

To the height of dams, again, there is a practical limit: earthen dams of great height require an enormous section, being consequently very costly as well as dangerous, and are in themselves difficult to manage as regards escape; masonry dams have a limit to their height, due to the pressure per unit of surface on the foundation; the highest yet built does not exceed 164 feet, and it is very improbable that that height will be exceeded for some time to come, unless iron is made to enter largely into their construction.

After choosing a site for a proposed reservoir, one of the first points requiring attention is the determination of its storage capacity up to different proposed levels of escape. For this purpose, marks are fixed at differences of level of about 5 or 10 feet, on any convenient short line of its section; and the contours of these levels marked out and surveyed all around the basin, in order to obtain the perimeters and areas at each contour, from which, as before shown, the contents of each lamina can be calculated, and the content up to any other contour. If, however, it be preferred to obtain this by means of a series of longitudinal and transverse sections taken up to the heights of the various contour levels, it is perhaps best to direct the former in conformity with the axis or axes of figure of the basin, and the transverse sections at right angles to them, and, as far as possible, at equal distances along them; although in many instances, unequal distances and inclined directions, more suited to the form and disposition of the ground, would give more correct results; and the inclined sectional areas, when multiplied by the cosine of the angle of obliquity, are easily reduced to the true values of their corresponding rectangular transverse sections. Should a winding river channel or depression form part of the basin, it is often more convenient and correct to estimate its content independently, and add it in afterwards.

The following are the three formulæ most used in obtaining the contents from the sectional areas :—

1. If there be only two sectional areas, A_1, A_2, taken at a time, at a common distance, *d*,

the contents $= \frac{d}{2}(A_1 + A_2)$, or $= \frac{d}{3}(A_1 + A_2 + \sqrt{A_1 A_2})$.

2. If there be three equidistant sections, A_1, A_2, A_3, taken at a time, and their common distance is *d*,

the contents $= \frac{d}{3}(A_1 + 4 A_2 + A_3,)$ prismoidal.

3. If there be any even number (n) of equidistant sections, A_1, A_2, &c., up to A_n, at a common distance, d, the contents $= d\left(\frac{A_1}{2} + A_2 + \&c.\ A_n - 1 + \frac{A_n}{2}\right)$.

The accuracy of result will of course depend on the closeness of the sections, and the suitability of their positions to the general form of the reservoir.

The capacity of the reservoir being obtained, the amount of supply that can be expected annually from the catchment area may be obtained, either in total quantities or in continuous quantities as cubic feet per second, by the aid of Parts 1 and 2 of Table II. of the Working Tables; in these calculations much labor is saved by deducting, in the first place, the allowance due to evaporation and absorption on the catchment area from the rainfall given, and making use of the available rainfall or rainfall run off as the basis of calculation for supply.

If a limited supply alone be required, the use of Part 1, Table III. of the Working Tables, will enable the contents of the reservoir, and extent of catchment area necessary to afford the supply to be rapidly determined. Part 2, Table III., may also be occasionally useful, where the supply is limited by the needs of an extent of land to be irrigated, or the population of a town requiring water for public purposes.

The section of waterway of escape has next to be determined; this depending on the flood discharge and the maximum downpour in twenty-four hours. In these calculations, Part 3, Table II. of the Working Tables is useful; so also are Parts 1 and 2, of Table IV., in connection with the formula already given for flood discharge.

The reduction or conversion of discharges or supplies into either total or continuous quantities for various intervals of time, can be rapidly effected by the aid of the Table of Equivalents, Table XI., Parts 1 and 2; and their conversion into other measures, English or metrical, may be facilitated by the use of Parts 5 and 6 of the same table.

All these are of course simply modes of calculating, or of shortening the calculation, of the quantities of water; the determination of them has to be left to the discretion of the engineer and the requirements of the case. Should the supply be required to maintain a certain depth of water for navigation in a canal, the seasons, the supply deficient, the loss in the canals from evaporation and filtration, and all such data, will determine the amount;—if for irrigation, the amount of land, its quality of soil, and probable water duty; on this latter subject information is given in Chapter III. and in the Hydraulic Statistics, in Part 2 of this Manual, where data of the waterings and water duty usual in France, Spain, Italy, and Northern and Southern India, are given.

If, again, the supply is required either for motive power or the public purposes of town supply, the amount and height of delivery require determining with reference to local conditions; with reference to this, therefore, no guide would be of use. Lastly, if the object is the control of floods, the whole of the physical conditions of the river and its banks, from its highest watershed down to its mouth or embouchure in the sea, will be matters affecting the amount, and the management and regulation of the storage.

5. DISCHARGES OF RIVERS, OPEN CHANNELS, AND PIPES.

The various modes of gauging velocities and discharges are described in the chapter on field operations and guaging. The calculation of velocity or of discharges, under different conditions and for different data, may be considered independently of gauging. It is important to the engineer that he should at any time be able to calculate, in a few moments, the discharge of any pipe, river, or canal, from such data as he may possess.

The number of calculated velocity formulæ, their variety, and the wonderful amount of complication in them, as well as the want of exactitude of result they give, is truly astonishing; and when, on the other hand, one observes some engineers adhering slavishly to the tables and data of one hydraulician, others to those of another, and others again going through the conscientious, but very lengthy, course of examining everything that every hydraulician has said or done in the matter of calculation of mean velocity of discharge, one cannot but feel pained as well as surprised.

It would be quite out of place in this portion of a Manual of this description, which has for its object the supplying the engineer with information and tables for calculating his quantities and data in as rapid a way as practical correctness will allow, to enter into a detailed investigation of all these formulæ, and the reasons for setting them all aside, and adhering to that adopted in preference, and to the exclusion of all others; it will, therefore, suffice for the author here to mention the reason for adopting any one formula or conclusion as it is brought forward. A comparison of the results of various hydrodynamic formulæ, will be given in Chapter III., among the miscellaneous detached paragraphs.

The general formula for discharge, based on the theories mentioned in the previous sections of this chapter, is

$$Q = A\,V = A\,(f g\,R\,S)^{\frac{1}{2}},$$

the terms of which are given in the general notation, page 10; the mean velocity of discharge being the smaller and more convenient quantity to deal with, for rivers and open channels, and the discharge itself being the quantity more often required for pipes, sewers, and closed tubes or tunnels of all sorts.

Taking, however, the expression for mean velocity of discharge, obtained by equating the accelerating effect of gravity down an inclined plane with the retarding effect of friction, it can be put into the form more convenient for English measures—

$$V = c \times 100\,(R\,S)^{\frac{1}{2}},$$

where c is a variable experimental coefficient, depending on the surface, the condition, the dimensions, and the hydraulic slope of the channel or pipe, and hence on a further experimental coefficient of fluid friction, and on a fresh development of the functions R and S: its value under extreme conditions varies from ·25 to about 2·00.

A correct formulated determination of the value of the coefficient, c, for all conditions, is a matter that can only be said to have been even approximately arrived at in the last few years, from an examination of the experimental results of d'Arcy and Bazin on the discharges of pipes, open channels, and ordinary rivers, and those of Humphreys and Abbot on the discharges of very large rivers, by Mr. W. R. Kutter, of Bern.

The determination of the coefficient, for which we are idebted to him, and tables rendering it easily found for open channels and rivers of any sort or dimensions, in metrical measures, are given in his valuable articles in the "Cultur Ingenieur" for the year 1870.

From these the values of the coefficient suited to English feet and cubic feet per second have been reduced; they are given in the table for coefficients of all sorts, Table XII., under the head of coefficients of velocity of discharge, in Part 3, pages lxxi. to lxxx.: these are also further explained by the table of coefficients of fluid friction in Part 1, Table XII., page lxix.

With the aid, therefore, of these tables of coefficients, and the values of the expression 100 $(R\,S)^{\frac{1}{2}}$, given in Table VII., pages xviii. to xxv., the values of V, the mean velocity of discharge of rivers and open channels can be rapidly determined in a few moments, according to the most improved and correct method yet known.

With the aid of the same tables of coefficients and the values of the expression,

$$Q = c \times 39{\cdot}27\ (S\,d^5) \text{ when } c = 1,$$

given in Table VIII., pages xxvi. to xxxvi., the actual discharge of any full cylindrical pipe, sewer, or tunnel, can also be determined.

These tables, to which explanatory examples are attached, can also be used for the converse purpose of obtaining the head, diameter, hydraulic slope or hydraulic radius, due to given discharges of channels and pipes; it will, however, be necessary for the calculator to remember that all dimensions, even diameters of pipes, are invariably kept in feet, and that all slopes are kept in the form known as the sine of the slope, mentioned in the general notation, page 11, of this chapter. Should it be necessary to reduce these from gradients given in other forms, such as in feet per English mile, or as a fall of unity to a certain length, Table VI., pages xiii. to xvii., will be found to save calculation.

So far for the velocity formula actually adopted, and the mode of working it in calculating results. As regards the formula itself, independently of the determination of the variable coefficient, it is none other but the Eytelwein formula, or Chezy formula, in a very much improved form, having the results of modern experiment incorporated with it. An examination of all the hydraulic formulæ for mean velocity shows that most, in fact, almost all of them, were modifications of the Chezy formula, some of them adding an additional term or function, and altering the value of the experimental coefficent, but still asserting its fixity. In the previous editions of this Manual, written before Mr. Kutter had published his valuable improvement, all these formulæ, having fixed coefficients, were rejected by the author, who at the same time asserted the principle that no fixed coefficient was suitable to all circumstances, and that the engineer should choose for himself a coefficient most suitable to the special circumstances, dimensions, and condition of the pipe, channel, or river, with whose discharge he was dealing; and that the results of experiments should be always consulted for this purpose.

A mode of successive approximation to the mean velocity was also recommended, first, assuming $c = 1$; and then from the mean velocity resulting, assuming a second value of c, according to the following table, a second true velocity of discharge was calculated.

v.	*c.*	*v.*	*c.*	*v.*	*c.*	*v.*	*c.*
1·0	·910	1·5	·960	2·0	1·000	2·5	1·023
1·1	·920	1·6	·968	2·1	1·005	2·6	1·026
1·2	·930	1·7	·976	2·2	1·009	2·7	1·030
1·3	·940	1·8	·984	2·3	1·014	2·8	1·033
1·4	·950	1·9	·992	2·4	1·018	2·9	1·037
						3·0	1·040

But these were intended to apply solely to canals in earth in good order. A few values of *c*, suitable to pipes under various velocities, were also given; but they were detached, and, from want of experiment, very insufficient. Yet the true state of the case, and the mode most advisable for adoption until investigations on a larger scale threw more light on the matter, was then clearly set forth.

Now that the experiments of d'Arcy and Bazin, of Humphreys and Abbot, and of Ganguillet and Kutter, have been comprehended in one formula, the labour of choosing a coefficient from experimental records is rendered entirely needless.

The determination or tabulation of this coefficient has gone through two stages of development. The first was that made by Bazin, based on the experiments conducted by d'Arcy, by Bazin himself, and by various engineers of the French Ponts et Chaussées, and is applicable to metrical measures. The principles asserted were that the coefficient depended on two quantities or qualities only, namely, the condition of surface of the bed and banks touched by the water, and the hydraulic mean radius of the section of discharge. Four categories of coefficients were adopted.

1st. For very smooth surfaces, well plastered surfaces in cement, and well planed plank.

2nd. For smooth surfaces, ashlar, brickwork, and ordinary planking.

3rd. For less smooth surfaces, as rubble.

4th. For earthen channels.

The values of the coefficient, A, being—

$$(1)\ 0{\cdot}00015\left(1 + \frac{0{\cdot}03}{R}\right)$$

$$(2)\quad 0{\cdot}00019\left(1+\frac{0{\cdot}07}{R}\right)$$

$$(3)\quad 0{\cdot}00024\left(1+\frac{0{\cdot}25}{R}\right)$$

$$(4)\quad 0{\cdot}00028\left(1+\frac{1{\cdot}25}{R}\right)$$

and the corresponding value of c for the English formula of discharge being $=\frac{1}{100\sqrt{A}}$ for metres, and $\frac{1{\cdot}81}{100\sqrt{A}}$ for English feet; the French formula for metres being

$$\frac{RS}{V^2}=A,$$

and the English formula for feet being

$$\frac{V}{100\,(R\,S)^{\frac{1}{2}}}=c.$$

The values of these coefficients, adapted to the corresponding formula in English feet, are generally as follows, in their respective categories:—

R.	C. (1)	C. (2)	R.	C. (3)	C. (4)
1·	1·41	1·18	1	0·87	0·48
1·5	1·43	1·22	2	0·98	0·62
2·	1·44	1·24	3	1·04	0·70
2·5	1·45	1·26	4	1·06	0·76
3·	1·45	1·26	5	1·08	0·80
3·5	1·46	1·27	6	1·10	0·84
4·	1·46	1·28	7	1·10	0·86
4·5	1·46	1·28	8	1 11	0·88
5·	1·46	1·29	9	1·12	0·90
5·5	1·46	1·29	10	1·12	0·91
6·	1·47	1·29	11	1·13	0·92
7·5	1·47	1·29	14	1·13	0·95
8·	1·47	1·30	15	1·14	0·96
19·	1·47	1·30	18	1·14	0·98
20·	1·48	1·31	20	1·14	0·98

To obtain the values of coefficients of mean velocity from the observed maximum velocity V_x, and values of R and S in English feet, we obtain from Bazin's formula $V_m = V_x - 14\sqrt{RS}$ for metres, which for English feet is

$$V_m = V_x - 23{\cdot}5\sqrt{RS},\ c = {\cdot}01\left[\frac{V_x}{\sqrt{RS}} - 25{\cdot}3\right].$$

Applying this coefficient to the formula $V_m = c \times 100\sqrt{RS}$, the true mean velocity of discharge V_m is obtained, and it is probable that this latter mode of determination is preferable both to the former and to the following method adopted by Kutter.

The second stage of development was effected by Kutter and Ganguillet; their own experiments on torrents and streams in Switzerland, combined with the results of Humphreys and Abbot on very large rivers, led them to believe that the coefficient should not be confined within so small a number of categories, and that also it was, besides being a function of the surface acted on by the water, and the hydraulic radius of the section, a function of the hydraulic slope.

They therefore extend the categories of coefficients suitable to open channels of all sorts in earthen beds into four distinct classes, and make some other additions to the categories adopted by Bazin; these new classes being ranged in accordance with the coefficient of fluid friction adopted as suitable to the surface under consideration.

A table of these general values of the coefficient of fluid friction is given in Part 1 of Table XII., page lxix.; and some local values from which the former were deduced by Mr. Kutter, are also given on the same page. The classes being determined by these means, the values of the coefficients of discharge are made to depend on them, as well as on the hydraulic slope and hydraulic radius of the open channel under consideration, and are obtained for metrical measures by the following expression :—

$$c_{,} = \frac{23 + \frac{1}{f} + \frac{0\cdot00155}{S}}{1 + \left(23 + \frac{0\cdot00155}{S}\right)\frac{f}{R^{\frac{1}{2}}}}$$

which is also given in the following form :—

$$c_{,} = \frac{z}{1 + \frac{x}{R^{\frac{1}{2}}}}$$

where $z = 23 + \frac{1}{f} + \frac{0\cdot00155}{S}$ and $x = f\left(23 + \frac{0\cdot00155}{S}\right)$.

The reduction of this expression for application to English measures, for which $c = 0\cdot0181\ c_{,,}$ is effected in pages lxxi. to lxxx. of the Working Tables; and if any convenient general value of f be assumed as applicable to the particular case, the coefficient corresponding to any ordinary values of R and S, likely to occur in practice on canals and rivers, can be read at sight.

The calculation of the discharge of pipes is conducted on exactly the same principle; although it is extremely unfortunate that the investigations of Ganguillet and Kutter were limited to open channels, and hence the application of his principles to pipes, though rationally superior to any other mode previously adopted, cannot be conducted with the same amount of experimental record in support, nor with the same amount of accuracy.

Assuming then the same formula for mean velocity of discharge—

$$V = c \times 100\ (R\,S)^{\frac{1}{2}},$$

and adapting it to terms of the diameter of a pipe in feet; it becomes for full cylindrical pipes and tubes of all sorts, where $R = \frac{d}{4}$

$$V = c \times 50\ (d\,S^{\frac{1}{2}}),$$

d as the actual discharge is the quantity more usually quired direct in the case of pipes, this is—

$$Q = A\,V = c \times \cdot 7854\, d^2 \times 50\,(d\,S)^{\frac{1}{2}},$$
$$c \times 39{\cdot}27\,(S\,d^5)^{\frac{1}{2}},$$

discharges in cubic feet per second.

The converse forms of this expression being—

$$d = \frac{1}{c^{\frac{2}{5}}} \times 0{\cdot}23 \quad \left(\frac{Q^2}{S}\right)^{\frac{1}{5}},$$
$$H = \frac{1}{c^2} \times 0{\cdot}0648\ \frac{Q^2}{d^5},$$

here H is the head in feet for a length of 100 feet, or is qual to 100 S.

The values of these quantities are given in Parts 1, 2, d 3, of Table VIII., for a value of $c = 1$, and the values c given in the table of coefficients of discharge, Table II., pages lxxi. to lxxiv., can be applied: the powers and ots of c can be taken from Part 7, Table XII.

With regard to these coefficients, it will be noticed that r want of sufficient experimental data, a coefficient of iction $f = 0{\cdot}010$ has been assumed as applicable to amelled or glazed metal pipes, and one of 0·013 for dinary metal and earthenware or stone-ware pipes under dinary conditions, but not new; and there is every ason to believe that these assumptions are generally rrect, if we compare the smoothness of surface of a lazed pipe with that of very smooth plaster in cement, d that of an ordinary pipe, in average condition, with at of ashlar or good brickwork.

In applying however, to pipes the coefficients of discharge, resulting from the formula of Mr. Kutter, one ould naturally be unwilling to push to extremes the inciple, asserted by him as applicable to open channels, d would prefer stopping at a point where the experiental data now forthcoming leave us. It would, therefore, seem imprudent at present to assume the law of coefficients asserted by Mr. Kutter, to hold good for a hydraulic radius R less than 0·1 feet; which, for falls steeper than 0·001 give as a coefficient for glazed pipes 0·84, and for ordinary pipes 0·61. This limiting hydraulic radius of 0·1 feet is that of a 5-inch pipe, or a pipe having a diameter of 0·4 feet; and we therefore assume for the present, and until further investigation has thrown more light on the subject, that the coefficient of discharge for all full pipes, having a diameter less than 0·4 feet, will be the same as for those of that diameter.

The above-mentioned modes of calculating the discharge of rivers, open channels, and full cylindrical tubes, are intended to apply generally.

It will, however, be perfectly evident that this does not by any means preclude the application of an allowance or deduction made for special circumstances. In actual fact, few channels are either perfectly straight, perfectly regular, or free from lateral and longitudinal irregularities; these alone may affect the amount of discharge by as much as five per cent., even after making allowance for loss of head by bends and obstructions; and the local conditions of a river, the wind, the amount of silt in suspension, the motion of its shoals, the change of the set of its currents, all seriously affect a discharge calculated from data that make no allowance for these circumstances.

For canals and regular rectangular and trapezoidal channels in earth in good order, calculated discharges will naturally give results more correctly than for natural or river channels; the errors due to various irregularities being very much reduced. The formulæ of discharge are, however, as frequently used in determining the section of canal intended to convey a certain discharge, as to obtain a discharge from data of an actual canal.

In these cases, a consideration of the various forms of section, suitable to different purposes, is also necessary. This matter has been treated and repeated in nearly the same terms in all works on hydraulics, and there is, perhaps, nothing new to be said about it; the entire omission of it in a Manual of this description might, however, be liable to cause disappointment; and hence the following remarks, most probably based on the ideas of Eytelwein and d'Aubuisson, though, perhaps, taken through other channels now forgotten, are therefore inserted for purposes of reference.

6. THE FORM OF OPEN CHANNEL

that will give a maximum discharge, is that which, for a given sectional area, has the least wetted border or perimeter, the semicircle, like the circle, is geometrically known to possess this property, and regular demipolygons externally tangential to the semicircle, have also more or less this property, according as their sections more or less approximate to it in form; the semicircle, too, has its hydraulic radius equal to half its middle depth, and this also holds for trapeziums of maximum discharge.

Hence Neville's geometrical construction for determining the form of the trapezoidal channel of maximum discharge that has given side-slopes and sectional area.

From the middle of the top width of the proposed trapezium, describe a semicircle with a radius, equal to the proposed depth, and draw the given slopes and the bottom tangential to it.

This form gives the top width = sum of the side-slopes,
the mean width = half the perimeter,

and the area $= d^2 \left(\tan \frac{B}{2} + \operatorname{cosec} B\right)$, where d = depth, and B = inclination of the slope with the horizon.

From these properties, the relative dimensions of trapezoids of maximum discharge, may be obtained for any side-slopes. They are given in the following table by Neville.

Relative Dimensions of Maximum Discharging Channels—(Neville).

		Factors for				
Slope.	**Angle.**	**Depth.**	**Bottom.**	**Top.**	**R.**	**Area.**
		In terms of the square root of the area.				
90°	0 to 1	·707	1·414	1·414	·354	$2d^2$
63° 26′	½ to 1	·759	·938	1·697	·379	$1·736d^2$
48° 34′	¾ to 1	·748	·675	1 996	·374	$1·784d^2$
45°	1 to 1	·740	·613	2·093	·370	$1·828d^2$
36° 52′	1⅓ to 1	·707	·471	2·357	·354	$2d^2$
33° 41′	1½ to 1	·689	·417	2·484	·345	$2·105d^2$
30° 58′	1⅔ to 1	·671	·372	2·608	·336	$2·221d^2$
26° 34′	2 to 1	·636	·300	2·844	·318	$2·472d^2$
Semicircle	curve	·798	·000	1·596	·399	$1·571d^2$
circle	curve	1·128	·000	·000	·282	$·785d^2$

These are most applicable in cases where heavy floods have to be provided for by a rapid drainage, and where the maximum discharge is the principal object.

For most practical purposes, however, such channels would be worse than useless, because depth is more expensive than width, because the high velocity generated might be destructive to the channel itself, and in cases where navigation is an object, the depth of draught would be too much affected by the fluctuation of supply; depth and velocity being thus limited, as well as the hydraulic slope, which is controlled by local circumstances, and the side-slopes, which depend on the nature of the soil, the width remains the only function of the section which admits of much variation.

Now, in a proportion of width to depth exceeding

14 to 1, which is about the lowest limit that will maintain a navigable depth, the side-slopes cease to remain a very important element, and the mean width can be dealt with equally well for rectangular and for flat trapezoidal sections; the practice in calculation, therefore, is, after assuming certain side-slopes, to reduce or increase the mean width by two or three feet at a time, until a safe bottom velocity is attained by the form of section thus approximated to, and the intended discharge thus arrived at. The next point is to know the relations between width and depth that give many sections that will discharge the same quantity with the same hydraulic slope. For this purpose their areas are inversely as the square roots of their hydraulic mean depths, and hence the square root of the cube of the channel sectional area, divided by the perimeter, must be constant. Thus:—

$$\left(\frac{w^3\,d^3}{w + 2d}\right)^{\frac{1}{2}} = m,$$

and hence $d^3 - \frac{2m^2}{w^3}\,d = \frac{m^2}{w^2}$.

Solving which, we obtain for a value of $w = 70$, and for convenient values of d up to 6, corresponding values of m. Thus:—

d	·25	·50	·75	1·00	1·25	1·50	1·75	2·00
m	8·7	24·6	45·0	69·0	96·9	126	158	193
d	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0
m	267	349	437	531	629	732	839	951

This equation being also worked out for the same values of d and other values of w, the results are formed into a table of equal discharging channel-sections, given in Part 4, Table XI., page lx., which answers all practical purposes in determining dimensions of section for open channels of any size, by applying multiples and submultiples to the dimensions there given. The table mentioned was taken from Stoddard's work, although there is also one very much like it in Neville's well-known work on Hydraulics, as there appeared to be no advantage in making a new one.

An additional table has, however, been made by the author to facilitate the determination of channels (not channel sections) of equal discharge, applicable to cases in which the variable coefficients of discharge, adopted by Mr. Kutter, are employed. Part 4, of Table XI., page lxi., gives a variety of depths, bottom widths, velocities, and hydraulic slopes, that are applicable to channels of one given discharge, and is useful in roughly determining dimensions and data necessary for various discharges.

The form of section of a pipe, with reference to its discharge, is a matter in which very little variation is practically possible: all small pipes being generally made cylindrical and kept constantly full. The quality of the interior surface of the pipe is however very important, the discharge being liable to be reduced as much as 33 per cent. by fouling and incrustation, the retarding influence being not so much the diminution of section as the increase of friction.

Formerly the method usually adopted in making allowance for incrustation consisted in reducing the diameter employed in calculating the discharge; the reduction being $\frac{1}{2}$ inch for pipes less than 3 inches in diameter, $\frac{3}{4}$ inch for 3-inch to 6-inch pipes, and 1 inch for pipes 6 inches to 1 foot and upwards in diameter. It is evident, however, that this principle is faulty, and that the reduction should be made for these circumstances in the coefficient of fluid friction employed in determining the coefficent of discharge. It is to be hoped also that in the future water pipes will not be allowed to fall into the disgracefully filthy condition that has too often existed in England, and that some enamelling or glazing process, like that of Dr. Angus Smith, will be more universally adopted.

It will be evident from an examination of the original formula, that in order to obtain a maximum discharge from a pipe, its hydraulic mean depth, R, must be a maximum. A full cylindrical pipe, having $R = \frac{d}{4}$ seems at first sight to be nearly perfect in this respect; and, under high velocities, doubtless gives the greatest scouring power;—but the segmental circular section, leaving an upper section, whose angle is 78½° empty, admitting of the advantage of making the upper segment movable for cleaning, gives a maximum discharge for nearly filled pipes under smaller velocities, as thus shown :—

	Segmental.	Full Circle.
Hydraulic radius	·6	·5
Velocity	1·095	1·
Discharge	1·026	1·

The egg-shaped section, usually adopted for sewers, is good for intermittent unfilled pipes, as it fills higher and flushes better :—one form is generally adhered to, in which the diameter of the bottom circle is half that of the top, and the depth of the sewer, and the radius of each side curve, are each equal to once and a half the diameter of the top circle; they are generally calculated for filling to two-thirds of their depth, and in that state their discharges and velocities bear well-known proportions to those of cylindrical sewers :—viz.

	Velocities.	Discharges.
Cylindrical, full	1·00	1·00
Ovoid, ⅔-full	1·04	·89
Cylindrical, ⅔-full	... ...	·61

Calculations connected with pipes and sewers may be sometimes shortened by taking discharges through pipes of the same section in proportion to the square of the head, and through pipes of the same head proportional to the square roots of the fifth powers of the diameters. In these, Part 7, Table XII., is of use.

In dealing with the slopes of pipes, it must be remembered that the hydraulic slopes are those that are dealt with in all formulæ of discharge. Pipes are usually placed two or three feet below ground, to protect them from frost, and follow its sinuosities, rarely being allowed to rise above the mean hydraulic gradient or slope: should they do so, a great loss of head results, unless air vessels are applied at those points, from which the air is allowed to escape through cocks every two or three days. As again it is comparatively rare that a single pipe is laid to any very great distance with a uniform fall, being more generally cut up into lengths having different falls, it becomes necessary to proportion the diameter of the pipe in these different lengths, so that the discharge may be that due to the smallest diameter. When with such a series of pipes of different diameters the total head is given, and the discharge is required, the case does not admit of direct solution, as each pipe must have its own proper head; in this case it is best to assume a discharge, and obtain separate heads due to it for each pipe in the series; the true heads, both total and separate, may be then obtained by proportion, and the gradients of each pipe, as well as the mean hydraulic gradient of the whole series (which is the slope that would be adopted for a single uniform pipe throughout) marked on the section of the design. The final discharge can then be calculated from any one of the pipes. An example of this is attached to Working Table, No. X.

7. OTHER THEORIES OF FLOW.

Before quitting the subject of flow and entering into that of velocities, it may be as well to mention two apparently more perfect, though far less simple, theories of flow, which have not yet brought about sufficiently extended practical results in the determination of discharges. The first is that of Dupuit: it neglects friction on the sides of the section of flow, thus considering motion in all vertical planes to be the same, and dealing with horizontal laminæ only; the surface lamina is considered to be in the condition of a solid gliding over an inclined plane, and each lamina below, except the bottom one, is urged on by its own weight and its cohesion to the upper lamina; the bottom fillet is retarded by its adhesion to the bed. Putting this in the form of an equation, summing, rejecting certain terms, integrating and applying three numerical coefficients, Dupuit obtains a result, which for English feet is—

$$v = \frac{S. R A.}{\cdot 08 W} - \cdot 082 + (\cdot 0067 + \cdot 9114 R S)^{\frac{1}{2}}.$$

It is this formula that has produced more correct practical results generally, than any one of the formulæ having fixed coefficients: next to it, in order of correctness, coming the Chezy formula, with a fixed coefficient $c = 1$. This theory assumes that the uppermost lamina moves invariably with the maximum velocity, which is not the case; the neglect of the friction of the banks might again not vitiate results if applied to large rivers or shallow channels; it is probable, therefore, that a modification of calculation suited to the facts more recently discovered, about maximum velocity, might render tl theory very perfect as well as practical. For more information, refer to Dupuit's "Etude Théorique et Pratique sur le mouvement des eaux couraútes, Paris, 1848," and Claudel's Tables, which contain extracts therefrom.

The second theory is that of the Mississippi survey, mentioned in the Mississippi Report, Philadelphia, 1861, which deduces the new formula mentioned, as giving the most correct results of all yet known; it is, however, unfortunate in its formulæ being rather inconvenient in some respects. While, therefore, the investigation and deduction of the formula is valuable on account of the information, and results of experimental data applied to it, the result is not so useful as regards the practical use of the formula, which was virtually set aside by the Mississippi Survey, whenever careful river-gauging was carried out in favour of other equations deduced from velocity observation.

In a work of this scope, it is impossible to go beyond the mere outlines of the demonstration adopted. Adopting the notation of the Mississippi Survey given at pages 11 and 12, it may be stated as follows.

The theory accepts uniform motion and the usually accepted application of the laws of uniform motion, but in retarding force, denies the stability of position of maximum velocity, and makes allowance for the resistance of the air on the water surface, as well as for the effect of wind.

The process of reasoning follows through the following equations.

The equations obtained for the forces, are as follows :—

(1). $lGgas = l(p + w)\,\phi\,\frac{U_o W + U_r p}{W + p}$

dividing both sides by Ggl,

putting $U_o = \cdot 93v + (\cdot 016 - \cdot 06f)\,(bv)^{\frac{1}{2}}$

$U_r = \cdot 93v + (\cdot 06f + \cdot 35)\,(bv)^{\frac{1}{2}}$

[redacted]

$$(2).\ \frac{as}{W+p} = \phi\left\{\cdot 93v + (bv)^{\frac{1}{2}}\,\frac{\left(W\left(\cdot 333 - \frac{d_1}{r}\right) + p\left(\frac{d_1}{r} - \cdot 667\right)\right)}{W+p}\right\}$$

putting W = qp, where q practically = 1 for large rivers.

$$(3).\ \frac{as}{W+p} = \phi\,(\cdot 93v + \cdot 167\,(bv)^{\frac{1}{2}} = \phi\,(z) = Cz^2.$$

$$(4).\ C = \frac{as}{(p+W)\,z^2}$$

by practical observation $C = \frac{s^{\frac{1}{2}}}{195}$, hence

$$(5).\ z = \left(\frac{195\,a\,s^{\frac{1}{2}}}{p+W}\right)^{\frac{1}{2}}$$

In this equation there are practically only four variables, a, p + W, s and z, once for ordinary natural channels p nearly = 1·015 W; hence if the values of any three are given, the fourth may be obtained, the transpositions of the equation being—

$$(6).\ s = \left(\frac{(p+W)z^2}{195a}\right)^2$$

$$(7).\ a = \frac{(p+W)z^2}{195\,s^{\frac{1}{2}}}$$

$$(8).\ p + W = \frac{195\,as^{\frac{1}{2}}}{z^2}$$

Now z is a variable, of which only two absolute values are known, viz., that for a rectangular cross section, and that for an ordinary river section, which are—

$$z = v + \cdot 167\ b^{\frac{1}{2}}\,v^{\frac{1}{2}}$$
$$z = \cdot 93v + \cdot 167\ b^{\frac{1}{2}}\,v^{\frac{1}{2}}$$

Substituting these in (5) and solving, we get for rectangular channels,

$$(9).\ v = \sqrt{\cdot 0064b + (195r_1\,s^{\frac{1}{2}})^{\frac{1}{2}} - \cdot 08b^{\frac{1}{2}})^2}$$

For ordinary river channels,

(10). $v = (\sqrt{\cdot 0081b + (225r_1 s^{\frac{1}{2}} - \cdot 09b^{\frac{1}{2}})^2};$

For large rivers, where $r > 12$ feet, and where $b = \dfrac{1\cdot 69}{(r + 1\cdot 5)^{\frac{1}{2}}} = \cdot 1856$, the first term may be neglected, and this latter equation becomes—

(11). $v = ([225r_1 s^{\frac{1}{2}}]^{\frac{1}{4}} - \cdot 0388)^2.$

If the discharge is known, and also two of the four variables in equation (5), provided they are not a and v, the other two variables may be computed by eliminating the unknown variable in the second member of that one of the transpositions of equation (11) whose first member is the variable sought, by substituting for it its value deduced from the equation (12),

$$v = \frac{Q}{a}.$$

No difficulty will be found in performing the calculation, except when s and $p + w$ are the known variables, in which rate an equation of a higher degree than the second cannot be avoided, and successive approximation must be adopted as follows:—

Assume a value of a, and find two values of v, one from equation (12), the other from (10) or (9), as the case may require; these values of v will not agree, hence assuming a new value for a, until the resulting values of v are identical.

An application of the above-mentioned Mississippi formulæ to the discharges of canals, or even of small streams and rivers, cannot by any means be considered satisfactory as regards result; although for large and very large rivers, the amount of exactitude resulting may exceed that of any other known formulæ.

8. VELOCITIES IN PIPES AND ARTIFICIAL CHANNELS.

The laws of the distribution of velocity in the section of an open channel, canal, or river, are not yet satisfactorily determined. A certain amount of knowledge has been deduced from observation of the variation of velocity in the vertical planes, but as regards that in the horizontal planes of the section, nothing has absolutely—and very little relatively—yet been determined. In pipes, on the contrary, the conditions of velocity are comparatively simple. All the valuable information on this subject, quoted in this work, is that deduced by d'Arcy and Bazin, and by Humphreys and Abbot, from the results of their extensive experiments.

The experiments of d'Arcy, in 1851, established the following law of velocity in full pipes :—

$$\frac{V - v}{\sqrt{RS}} = 11{\cdot}3 \left(\frac{r}{R}\right)^{\frac{3}{2}},$$

This equation is in terms of metrical measures—

V = central velocity.

v = the velocity anywhere at a distance = r from the centre.

R = the radius of the pipe.

S = the loss of head or slope per running metre.

This equation in another form becomes—

$$V - v = \frac{11{\cdot}3}{R} r^{\frac{3}{2}} \sqrt{S}.$$

This formula was deduced by d'Arcy from observations taken at from one-third to two-thirds of the radii of various pipes from the centre; beyond $\frac{2}{3}$ of the radius, it is probable that the law does not hold good, and that the decrement of velocity should be more rapid than t indicated by the formula. Under any circumstances, however, it is clearly established that the velocities in a full cylindrical pipe, are equal at all points equidistant from the centre, and that the above law of decrement holds good for the central ⅔ of the diameter taken in any direction. In a pipe of rectangular section, the velocities are equal at any four points, taken symmetrically with reference to the centre of figure.

In open channels, however, this almost mathematical accuracy is entirely absent, and the perturbations produced near the surface of the water does not allow us to hope that any formula can be arrived at, which would give the actual velocity at any point in terms of the mean velocity and the co-ordinates determining the position of that point. These perturbations appear to be more considerable in proportion to the diminution of velocity, and the increase of depth of channel, and are coincident with a depression of the locus of maximum velocity; in the extreme cases, the curves of equal velocity in the section cut the surface of the water very obliquely.

The following are the conclusions drawn by Bazin on this subject:—

1st. For a very wide rectangular channel—

$$\frac{\mathrm{V} - \mathrm{v}}{\sqrt{\mathrm{H\,S}}} = \mathrm{K}\left(\frac{h}{\mathrm{H}}\right)^2,$$

where V = central velocity at the surface.
v = velocity at a point at a depth h below it.
H = total depth of water.
S = hydraulic slope of the water surface.

The above law of velocity is proved to hold good for very wide channels; the cases under experiment give a practically constant value of K = 20·0, the extremes varying between 15·2 and 24·9;—it would also appear that for a

[redacted]

tangular canal of infinite width, in which the influence the sides was entirely made to disappear, K would 24·0.

When, however, the depth of a rectangular channel is at enough, in proportion to the breadth, to make the luence of the lateral walls show itself in the middle of current, this law does not hold, nor does any law of crement of velocity seem possible, and mere generali- ions, in terms of the mean velocity, can alone be rived at.

If, then U = the mean velocity in a canal, the section which is very great in proportion to its depth—

$$U = \frac{1}{H}\int_{0}^{H}\left[V - K\left(\frac{h}{H}\right)^{2}\sqrt{RS}\,\right]dh$$

$$= V - \frac{K}{3}\sqrt{RS}$$

d the depth h below the surface is determined by the pression $\left(\frac{h}{H}\right)^{2} = \frac{1}{3}$; whence $h = 0·577\ H$, which is, in t, saying that the mean velocity is found at about $\frac{3}{5}$ of e total depth. This, however, assumes the before-men- med parabolic law of the decrease of velocity in each rtical plane, a hypothesis only admissible in a very large d perfectly regular canal.

In fact, however, and from experiments quoted, it pears that the locus of mean velocity is often below of the depth, and more often below $\frac{3}{4}$ of it; and that ıen the depth of the canal is great, and the velocity ble, the curve of mean velocity approaches still nearer e bottom, and goes as low as $\frac{4}{5}$ of the depth.

Taking the above relation $U = V - \frac{K}{3}\sqrt{RS}$, where $\frac{\sqrt{RS}}{U}$ $\sqrt{A}$, and $K = 24·0$, for a channel of infinite width; in is case also we get $\frac{V}{U} = 1 + 8\sqrt{A}$, as a result applicable to this special case, which supposes the parabolic law applicable throughout the whole breadth of the channel; and this differs greatly from the results of experiment on channels, which gives $\frac{V}{U} = 1 + 14 \sqrt{A}$.

The locus of maximum velocity is, however, not always at the centre of the surface, but is at a greater depth in proportion as the depth of the canal is greater and the mean velocity is less, being sometimes as low as $\frac{2}{5}$ the total depth.

The determination of bottom velocity can, in rectangular canals, be alone made in the special case of one supposed to be of infinite breadth: for this case, putting $h = H$ in the original formula, we obtain the velocity $w = V - K \sqrt{RS}$; but in all other cases no law can be given. The greatest of bottom velocities is in the middle and the least at the sides.

The velocity along the vertical sides of a rectangular canal, is generally greater in the middle than at the top or at the bottom; but beyond this fact, the determination of the exact velocity at any point of the side remains a very difficult problem yet unsolved.

The laws of velocity in canals of semicircular section are far less complicated than those of rectangular section:—the law of decrement of velocity is expressed in the following formula:—

$$\frac{V - v}{\sqrt{RS}} = 21 \left(\frac{r}{R}\right)^3$$

the extreme values of the coefficient deduced from experiment being 18·2 and 23·2; and the terms of the expression being similar to those in the equation for decrement of velocity in sections of pipes before mentioned:—

[redacted]

f in this we make $r = R$, we obtain as for rectangular hannels, the bottom velocity, $w = V - 21\sqrt{RS}$.

And the mean velocity will be deduced thus :—

$$U = \frac{1}{\pi R^2}\int_0^R \left[V - K\sqrt{RS}\left(\frac{r}{R}\right)^3\right] 2\pi r . dr$$

$$= V - \tfrac{2}{5} K\sqrt{RS}; \text{ where } \frac{\sqrt{RS}}{U} = \sqrt{2A};$$

$$\text{hence } \frac{V}{U} = 1 + \tfrac{2}{5} K\sqrt{2A}; \text{ where } K = 21$$

$= 1 + 11{\cdot}9\sqrt{A}$: an equation differing but little from that deduced from experiment on semicircular canals.

The radius $r_{,}$, of the circle of mean velocity of the section $= R . \sqrt[3]{\tfrac{2}{5}} = 0{\cdot}737\ R$;—which is saying that this is at about three-quarters of the radius from the centre, whereas in fact it is farther.

Taking finally the two expressions for decrement of velocity in canals of rectangular and semicircular section,

$$\frac{V - v}{\sqrt{HS}} = K\left(\frac{h}{H}\right)^2 \text{ and } \frac{V - v}{\sqrt{RS}} = K\left(\frac{r}{R}\right)^3$$

a general expression may be deduced from them,

$$V - v = \phi\sqrt{RS};$$

and as under these circumstances absolute velocities cannot be dealt with, it is better to make use of relative velocities, and by dividing each side of the general equation by U to transform it into the form

$\dfrac{V - v}{U} = \phi\sqrt{A}$; which is therefore true for all canals where ϕ is a function of the relative (not of absolute) co-ordinates determining the position of the point whose velocity is under consideration, their values being taken in proportion to the dimensions of the section.

With regard to velocities in natural and artifici channels generally, by far the most important result arrived at by d'Arcy and Bazin, is the relation between the maximum velocity and the mean velocity of discharge, represented by this equation, suitable to metres:

$$\frac{V}{U} = 1 + 14\sqrt{A};\ \text{and since } A = \frac{RS}{U^2};\ V - U = 14\sqrt{RS};$$

these equations reduced to English measures become

$$\frac{V}{U} = 1 + \frac{25{\cdot}34}{c \times 100};\ \text{and } V - U = 25{\cdot}34\sqrt{RS}.$$

The advantage derived from the application of this law in gauging is probably greater than that of any other velocity discovery of modern times.

Velocities in Natural Channels.

The laws of variation of velocity in horizontal planes, with reference to different forms of section have not yet been satisfactorily deduced, such velocities have therefore to be determined locally when required; the horizontal curves of velocity again vary much in different stages of the river or stream under consideration; the records therefore of such velocities involve much labour, and have not yet shown themselves of sufficient practical importance to repay the labour and trouble of their observation. The laws of variation of velocity in vertical planes have been most fully investigated by Captains Humphreys and Abbot on the great Mississippi Survey.

It was previously generally believed that the maximum velocity of any river or channel was that on the surface in the middle; that the mean velocity varied between ·7 to ·95 of the maximum velocity, in natural channels, and was generally ·8 for rectangular sections; and that the bottom velocity equalled twice the mean velocity less the maximum velocity, or ·6 of the maximum velocity for rectangular sections. There were also numerous other equations of relation between these quantities given by various theorists, none of them probably more correct than the above.

There is every reason to believe that this subject, difficult in itself, has been rendered more difficult to manage from the falsification of results by using many different complicated instruments, possessing inherent errors, and not admitting of a just comparison; the Mississippi observations being conducted on a very large scale, and in the simplest manner possible, have brought forth very important results. From their experimental data it has been deduced that the velocities at different depths below the surface in a vertical plane, vary as the abscissæ of a parabola, whose axis is parallel to the water-surface, and may be considerably below it, thus proving the maximum velocity to be generally below the surface; the equation of this curve with reference to its axis, taking the depths, relatively to the total depth, as ordinates, was obtained in the form—

$$y^2 = 1{\cdot}2621\ D^2 x$$

where D = total depth of bed below the surface, and x and y are the co-ordinates to the axis.

They also deduced that if d is the depth of the axis of the parabola, or locus of maximum velocity from the surface, then

$$d_1 = ({\cdot}317 + {\cdot}06 f)\, R$$

where R = hydraulic mean radius, and f = force of wind taken positive or negative, and taken = 1 when the velocity of the wind and current are equal, and = 0 for a cross wind or calm.

The following are other important equations, with regard to velocity in vertical planes, that they deduced, which though they are not so useful practically as might be wished, are inserted here for reference.

(For symbols refer to the notation given in the paragraph on that subject.)

Formulæ for velocity in any vertical plane:

(1) $b = \frac{1{\cdot}69}{(D + 1{\cdot}5)^{\frac{1}{2}}} = {\cdot}1856$ only when $D \; 7 \; 30$ feet.

(2) $d_1 = ({\cdot}317 \times {\cdot}06f)\,D$ very nearly.

(3) $V = Vd_1 - (bv)^{\frac{1}{2}}\left(\frac{d - d_1}{D}\right)^2$

(4) $V_o = Vd_1 - (bv)^{\frac{1}{2}}\left(\frac{d_1}{D}\right)^2$

(5) $V_D = Vd_1 - (bv)^{\frac{1}{2}}\left(1 - \frac{d_1}{D}\right)$

(6) $V_m = \frac{2}{3} Vd_1 + \frac{1}{3} V_D + \frac{d_1}{D}\left(\frac{1}{3} V_o - \frac{1}{3} V_D\right)$

(7) $V_{\frac{D}{2}} = V_m + \frac{1}{12}(bv)^{\frac{1}{2}}$

(8) $V_{d_1} = V_m + (bv)^{\frac{1}{2}}\left(\frac{1}{3} + \frac{d_1(d_1 - D)}{D^2}\right)$

(9) $V = V_m + (bv)^{\frac{1}{2}}\left(\frac{D(\frac{1}{3}D - d_1) + (2d_1 - d)}{D^2}\right)$

in which equation (9) is a mere combination of equations (3) and (8).

For velocity in the mean of all vertical planes the following formulæ have been deduced:

(1) $b = \frac{1{\cdot}69}{(r + 1{\cdot}5)^{\frac{1}{2}}}$.

(2) $d_1 = ({\cdot}317 + {\cdot}06f)\,r$.

(3) $U_m = {\cdot}93v$.

(4) $U = {\cdot}93v + \left(\frac{dr({\cdot}634 + {\cdot}12f) - d^2}{r^2} - {\cdot}06f + {\cdot}016\right)(bv)^{\frac{1}{2}}$.

(5) $U_o = {\cdot}93v + ({\cdot}016 - {\cdot}06f)\,(bv)^{\frac{1}{2}}$.

(6) $U_r = {\cdot}93v\,({\cdot}06f - {\cdot}35)\,(bv)^{\frac{1}{2}}$.

(7) $Ud_1 = {\cdot}93v + \{[{\cdot}317 + {\cdot}06f]^2 - {\cdot}06f + {\cdot}016\}\,(bv)^{\frac{1}{2}}$.

(8) $v = \left([1{\cdot}08\,U_{\frac{r}{2}} + {\cdot}002b]^{\frac{1}{2}} - {\cdot}045b^{\frac{1}{2}}\right)^2$.

The most important result of all these data and deduc-

[redacted]

ons is the following, a fact of great practical use in uging rivers, that the ratio of the mid-depth to the ean velocity in any vertical plane is independent of the idth and depth of the stream (except for an almost appreciably small effect) absolutely independent of the epth of the axis of the curve before referred to, and early independent of the mean velocity The formula xpressing this is

$$(7)\ V_{\frac{D}{2}} = V_m + \frac{(bv)^{\frac{1}{2}}}{12},\ \text{where}$$

V_m is the mean velocity on any curve in the vertical plane.

$V_{\frac{D}{2}}$ is the mid-depth velocity.

v is the mean velocity of the river.

D is the depth of the river at the spot.

$b = \frac{1 \cdot 69}{(D + 1 \cdot 5)^{\frac{1}{2}}}$, which when $D > 30$ ft. $= \cdot 1856$.

The application of this result to gauging is shown in Chapter II. on Field Operations.

9. BENDS AND OBSTRUCTIONS.

The irregularities of a river materially affect its velocity; the following remarks on this subject, by Captains Humphreys and Abbot, are instructive on this point.

"Even on a perfectly calm day, there is a strong "resistance to the motion of the water at the surface, "independent of, and not mainly caused by the friction "of the air; the principal cause being a loss of force, "arising from the upward currents or transmitted motion "caused by the irregularities at the bottom. There is "also an almost constant change of velocity at various round the bend;—it is, however, always assumed that each angle is one of 30°, and the effect is estimated as due to the number n of such bends or deflections of 30°; and this enables the formula to be put into the simpler form—

$$h_{,} = \frac{n\,V^2}{536} = n\,V^2 \times 0{\cdot}001865.$$

The values of this formula, for various velocities and bends, are given in Part 2, of Table X., page li., and an explanatory example at page lii.

A formula more suited to bends of pipes, is that of Weisbach; it is for cylindrical pipes—

$$h_{,} = \frac{a}{180'}\,\frac{V^2}{2g} \times \left\{ {\cdot}131 + 1{\cdot}847 \left(\frac{r}{R}\right)^{\frac{7}{2}} \right\}$$

and for rectangular tubes—

$$h_{,} = \frac{a}{180^\circ},\frac{V^2}{2g} \times \left\{ {\cdot}124 + 3{\cdot}104 \left(\frac{d}{2R}\right)^{\frac{7}{2}} \right\}$$

but as the bends of pipes, known as quarter bends, are generally taken as 90°; the factor—

$$\frac{a\,V^2}{180^\circ \times 2g} \text{ becomes } = \frac{V^2}{128{\cdot}8} = {\cdot}007764.$$

In this formula r and R are the radii of the pipe and of the bend, and the other terms are as before. The loss of head due to bends in pipes is, however, generally required as corresponding, not to mean velocities of discharge, but to the discharges themselves. The values given by this formula have, therefore, been tabulated in this form, and are given in Part 1, of Table X., page l.; an explanatory example is also attached.

The ordinary formula for calculating the rise in feet resulting from an obstruction in the section of a river channel, is that of Dubuat; it is—

$$h_{,,} = \left(\frac{V^2}{m^2.\,2g} + S\right) \left\{ \left(\frac{A}{a}\right)^2 - 1 \right\}$$

where A, a, are the normal and the reduced sectional areas,

S is the sine of the hydraulic slope of the river,

and m is the experimental coefficient.

Now, as in most cases, S is less than ·001, that term may be neglected, and taking $m = \cdot 96$, $m^2 = \cdot 92$, and the formula becomes—

$$h_{,,} = 0\cdot 0169 \, V^2 \left\{ \left(\frac{A}{a}\right)^2 - 1 \right\}$$

The values of this are given in Part 3, of Table X., page li., and an explanatory example on page lii.

10. DISCHARGE FROM ORIFICES AND OVERFALLS.

The discharge from orifices and overfalls, which to the hydraulic engineer generally resolve themselves into sluices and weirs, is a subject that was fully entered into by hydraulicians of past times, and to which very little information has been added by recent experimentalists. Nor is it by any means likely that further contributions will be soon made to this branch of hydraulic science, as there have recently been to that of the discharges of open channels; the practical interest attaching itself to the exact determination of discharge of a sluice or a weir, not being in excess of the amount of exactitude already attained. All accepted information on this subject being to be found, with but little variation, in the older books, the author has had little choice left to him, and has therefore taken the following notes almost entirely from Bennett's translation of d'Aubuisson's hydraulics.

Setting aside the experiments of the more ancient philosophers, and assuming that the discharge from any orifice is

$$Q = AV = A.\, m\, \sqrt{2g\,H}$$

where H = the head of pressure of the orifice,

m = the coefficient of reduction obtained by experiment,

V = the mean velocity of discharge,

and the pressure being supposed to be kept perfectly constant, the first of the more modern hydraulicians to obtain experimental values of m, on a scale larger than the previous very petty experiments, was Michelotti. His experiments conducted at Turin in 1767, under heads of pressure up to 22 feet, determined coefficients of reduction varying from 0·615 to 0·619, for circular orifices, up to 6½ inches in diameter, and coefficients varying from 0·602 to 0·619 for square orifices, up to 3 inches in length of side. The next important experiments did not so much include increase of head as increased dimension of opening. Messrs. Lespinasse and Pin, Engineers of the Languedoc Canal, 1782 to 1792, made experiments on rectangular openings, or sluices 4·265 feet broad, and having heights varying from 1·575 to 1·805 feet, under heads on their centres of, from 6·2 to 14·5 feet; the coefficients deduced varied from ·594 to ·647, the mean being 0·625; they also observed that the discharge from two sluices opened at one time side by side, was not double that from one sluice. The next important experiments were those of Poncelet and Lesbros, at Metz, in 1826; they deduced a law for the determination of coefficient of discharge of rectangular orifices under various proportions of head of pressure and depth of opening to width; these coefficients reduced by Rankine are given in a tabular form in Part 4 of Table XII., at page lxxxii. of the working tables. The next important experiments recorded were those conducted by M. George Bidone, at Turin, in 1836, on orifices on parts of which the contraction was suppressed, the extreme of suppression being a case in which the whole of the contraction was suppressed by fitting an interior short tube to the mouth of the orifice: his resulting formula of discharge was for rectangular orifices—

$$Q = m \, A \, \sqrt{2gH} \, (1 + 0{\cdot}152 \, \tfrac{n}{p})$$

and for circular orifices,

$$Q = m \, A \, \sqrt{2gH} \, (1 + 0{\cdot}128 \, \tfrac{n}{p})$$

where n is the portion of the perimeter p, whose contraction is suppressed.

About this time also some further experiments were made by Castel and d'Aubuisson; and some by Borda on orifices in sides not plane.

The results of all these experiments show that the extreme limits of the value of m, are 0·50 and 1·00 for orifices in all sorts of sides, and under all conditions, and are 0·60 and 0·70 for orifices in plane sides: also that the general mean value of m for orifices in a thin plate is 0·62; this, however, is perhaps more true for small circular orifices than for any other class of them. In this case therefore

$$V = 0{\cdot}62 \times 8{\cdot}025 \sqrt{H} = 4{\cdot}975 \sqrt{H},$$

and for rectangular orifices of a similar class, the values of m, ranging from 0·572 to 0·709 given at page lxxxii., must be applied to the general formula

$$V = m \times \sqrt{2gH}$$

in order to determine the mean velocity of discharge, which when multiplied by the sectional area gives the quantity discharged per second.

In the special case in which the reservoir of supply, still being kept at a constant level, is seriously affected by e velocity of the water supplying it, the discharge of e orifice will be augmented on this account, and then

$$V = m\sqrt{2g\left(H + \frac{W^2}{2g}\right)} = m\sqrt{2gH + W^2},$$

here W = the initial velocity of entrance.

For the special cases in which an open canal is attached o the orifice at its exit, in such a manner that the sides nd bottom of the canal are continuations of those of the ifice, the coefficient of contraction remains the same, xcept when the head on the orifice is less than $2\frac{1}{2}$ times he height of the orifice: in this latter case the coefficient ay have to be materially reduced. An extreme case iven by Poncelet and Lesbros, being one of a discharge hrough an orifice 0·164 feet high, under a head of 0·118, ave a value of $m = 0·452$, while without an attached hannel the value of m was $= 0·612$: further, when the vel of the attached channel was exactly at the same vel as the floor of the reservoir of supply, the value of m as reduced to 0·443. The law of reduction of coefficient ecessary for these cases is not yet given in a definite form.

The inclination of the attached channel, when less than e in 100 did not affect the coefficient in any way, but hen increased to one in 10, had the effect of increasing he coefficient from 3 to 4 per cent.

The above includes all the general deductions about ifices that are likely to be of any use to the engineer; more practical collection of coefficients of discharge for ifices is given in Part 4 of Table XII., at pages lxxxi. nd lxxxii.; and the value of the expression

$$V = m\sqrt{2gH}$$

given, for various heads, and for all the values of m that re commonly used in Table IX., pages xxxvii. to xlviii.; me explanatory examples also follow that table.

It may be observed, however, that although the minutiæ of discharges under certain experimental conditions have been sedulously preserved, there is yet considerable doubt what coefficient should be used for the larger sluices or openings that occur in practice. It is no doubt unfortunate that experimentalists should differ, but at the same time the circumstances under which the amount of discharge from a sluice is an important consideration only occur generally to those who are capable, and have the opportunity of determining it accurately by experiment themselves.

The ordinary coefficient for a sluice of moderate size, for small lock or dock-gates, or mill-gates, is generally taken at 0·62: that for a narrow bridge-opening, which may be considered as a large sluice, at 0·82; and that for very large well-built sluices, large wide openings out of reservoirs continuing at a level with the bottom of the reservoir, and large bridge-openings of the modern type, at 0·92.

The term H, representing the effective head of pressure, is differently estimated in various cases: in ordinary cases of sluices, supplied from a reservoir above them, the head is the difference of level between the surface of the water in the reservoir and the centre of figure of the sluice; but when the sluice is drowned, that is, has a perceptible depth of water standing below its exit, but above the sluice itself, the head is the difference of level of the water above and of that below it; in bridge-openings also, the head is the difference of water level above and below the bridge.

The most recent experimental determination of coefficients of discharge for head-sluices supplying small channels is that of d'Arcy and Bazin; the results of these operations will be given with the account of the mode of gauging adopted by them in Chapter II.

Orifices with mouthpieces attached were even in the time of the Romans known to have a greater discharge than without them. In order to effect this increase it is, however, necessary that the length of the attached or additional tube should be twice or three times the diameter of the orifice, otherwise the fluid vein does not entirely fill the mouth of the passage. The experiments of Michelotti and Castel determined a mean coefficient of discharge for cylindrical mouthpieces of 0·82, the extremes being 0·803 and 0·830; the singular effects produced under some circumstances by the application of cylindrical mouthpieces are more curious than useful. Conical converging mouthpieces increase the discharge more highly: the experiments on them of Castel, engineer of the waterworks of Toulouse, are exceedingly interesting; they demonstrated that under varied heads the coefficients of discharge and of velocity were practically constant for the same mouthpiece, and that for the same orifice of exit the coefficient of discharge increased from 0·83 for a cylindrical mouthpiece in proportion to the increase of the angle of convergence of the mouthpiece employed up to 0·95 for an angle of 13½°; and that beyond this angle the coefficient of discharge diminishes to 0·93 for 20°, and afterwards decreases more rapidly. The length of mouthpiece employed in these cases as well as in the former was 2½ times the diameter of the orifice. Some experiments by Lespinasse on the canal of Languedoc showed the enormous increase of discharge effected by using converging mouthpieces: his mouthpieces were truncated rectangular pyramids 9·59 feet long, the dimensions at one end 2·4 × 3·2 feet, at the other ·44 × ·62 feet, and were used in mills to throw the water on to water wheels; their opposite faces were inclined at angles of 11° 38′ and 15° 18′, and the head employed was 9·59 feet; the experiments resulted in determining a coefficient of discharge varying from 0·976 to 0·987.

Conical diverging and trumpet-shaped mouthpieces still further increase the discharge from an orifice: the experiments of Bernouilli, Venturi, and Eytelwein have thrown much light on this subject, and showed the coefficient to lie between 0·91 and 1·35. Venturi concluded that the mouthpiece of maximum discharge should have a length nine times the diameter of the smaller base, and a flare of 5° 6′, and that it would, if properly proportioned to the head of pressure, give a discharge 1·46 times the theoretic unreduced discharge through an orifice in a thin side.

Overfalls and Weirs.

An overfall may be considered to be a wide rectangular orifice in an ultimate position, where the head on the upper edge is zero; and its discharge may be therefore computed in the same manner as that of an orifice.

The discharge of an orifice is according to the parabolic theory—

$$Q = m \times \tfrac{2}{3} \sqrt{2g} \times w \left(H \sqrt{H} - h \sqrt{h}\right)$$

where h and H are the heads on the top and bottom edge, and d and w are the depth and width of the orifice; if then H = mean head on the centre of the orifice, and the orifice becomes an overfall, this formula becomes

$$Q = m \times \tfrac{2}{3} \sqrt{2g} \times w \left\{\left(H + \frac{d}{2}\right)^{\frac{3}{2}} - \left(H - \frac{d}{2}\right)^{\frac{3}{2}}\right\}$$

developing this, and putting wd = A, the sectional area,

$$Q = m \, A \, \tfrac{2}{3} \sqrt{2g H} \left(1 - \frac{d^2}{96 h^2}\right)$$

and as d is comparatively small, the last term is = O, hence

$$Q = m \, A \, \tfrac{2}{3} \sqrt{2g H}; \text{ and } V = m \, \tfrac{2}{3} \sqrt{2g H}$$

where H is the head on the sill of the overfall.

[redacted]

The value of the coefficient, *m*, varies according to the m of overfall. It was determined by M. Castel, at ulouse, by a large series of experiments: and also by ancis, in the Lowell experiments referred to in Chapter , on Gauging.

The experiments of M. Castel showed that, for the urate employment of a general coefficient of discharge, dimensions and conditions of an overfall should fall hin one of the three following classes.

1st. When the length of the overfall sill extends to entire breadth of the channel, and the head on the sill less than one-third the height of the dam or barrier, the fficients remain remarkably constant, varying only from 64 to 0·666. Hence generally for this case, $m = 0{\cdot}666$.

2nd. When the length of the overfall sill is less than entire breadth of the channel of supply, but is greater n a quarter its breadth, the coefficient lies between two extremes of 0·666 and 0·598, and is strictly dedent on the ratio of the length of sill to breadth of nnel;—hence it is for the following relative breadths:

ative breadth.	Coefficient.	Relative breadth.	Coefficient.
1·00	0·666	·50	0·613
·90	0·658	·40	0·609
·80	0·647	·30	0·600
·70	0·635	·25	0·598
·60	0·624		

3rd. If the length of the overfall sill is equal, or even y nearly equal, to one-third the breadth of the channel, coefficient remains very constant, varying only between 9 and 0·61. Hence generally for this case, which is parularly favourable for gauging small streams, $m = 0{\cdot}60$.

In other cases, that is, when the length of the sill is s than a quarter the breadth of the channel of supply, the coefficient depends on the absolute length of sill, and requires determining specially: it increases from 0·61 to 0·67 in direct proportion to the diminution of absolute length of sill.

With reference to the three cases suitable for practical purposes, the experiments of M. Castel showed that when the sectional area of the overfall was less than one-fifth of that of the normal section of the channel of supply, the effect of velocity of approach in the channel did not modify the value of the coefficient: for other conditions, the modification necessary was not determined in a very satisfactory form:—the new equation for mean velocity of discharge being changed from

$$V = m \tfrac{2}{3} \sqrt{2g H}$$

into

$$V = m \tfrac{2}{3} \sqrt{2g (H + \cdot 035\, W^2)}$$

where W = the surface velocity of approach, not determined from observation, but from its assumed ratio to the mean velocity, perhaps therefore the modification of the coefficient, m, by other authors into a new coefficient

$$m_1 = m \left\{ \left(1 + \frac{h}{H}\right)^{\frac{3}{2}} - \left(\frac{h}{H}\right)^{\frac{3}{2}} \right\}$$

where h is the head due to the velocity of approach, and H is the head on the weir sill, is a preferable arrangement.

For the special cases in which channels are attached in continuation of the sides of the overfall, the coefficients in the experiments of Poncelet and Lesbros were reduced by 18 to 33 per cent. If, however, the fall to the channel is more than 3 feet, no reduction is generally made in the coefficients.

It may be noticed that the head on the sill used in the above expression is that in the centre of the overfall, which is independent of the rising of the water at the wings, a phenomenon to be observed in almost all cases of weir discharges.

In all the above cases, it is supposed that thin edges, as of metal sheets, or one-inch waste-boards, are used; for broad or round lopped crests, the coefficients will require reduction. See the coefficients given in Part 5, of Table XII., page lxxxiii.

Obstructed Overfalls.—When obstacles occur on the sill of an overfall, as dwarf pillars or blocks, a deduction in the discharge over the sill is made not only on account of the reduction of section, but on account of the contractions resulting. Francis's formula is applicable to these circumstances in cases where the length of weir sill equals or exceeds the head;—it is

$$Q = \tfrac{2}{3} m \sqrt{2g} \cdot (l - 0{\cdot}1\, n\, H)\, H^{\frac{3}{2}}$$

where n = the number of end contractions,
= 2· when there is no central obstruction,
l = length of weir sill,
$l\,H$ = A the sectional area of discharge,
and m = 0·6228.

In case the weir sill has the same breadth as the channel of supply, $n = 0$; and in that case

$$Q = 3{\cdot}332\, l\, H^{\frac{3}{2}}.$$

This, it will be observed, varies from that of Castel, which, under the same conditions, gives $Q = 3{\cdot}557\, l\, H^{\frac{3}{2}}$.

Partly Drowned Overfalls.—When a weir has its tail water above the edge of the sill, it may be treated as a combination of an overfall with an orifice; the upper portion down to the level of the lower water as an overfall, and the lower portion from that down to the sill level as a rectangular orifice, and the discharges calculated separately for each. Using, however, the same value of H in both cases, H being the head due to the overfall, that is, down to the level of the tail-race.

Some further values of coefficients of weir discharge are given in the accounts of gauging in Chapter II. To aid in the computation of discharges from overfalls, the velocities of discharge due to various heads and various coefficients may be obtained from those given in Table IX., pages xxxvii to xlv., by reducing the velocities there given by one-third; the results multiplied by the section of overfall are then the required discharges. The method thus adopted enables the same table to be used in computing the discharges of both orifices and overfalls. A table of weir coefficients is given on page lxxxiii., and some explanatory examples on pages xlvi. to xlviii.

11.—EFFLUX OR DISCHARGE FROM PRISMATIC VESSELS OR RESERVOIRS.

The following formulæ given by d'Aubuisson may be considered useful for reference in the cases in which they are required in engineering practice:—

First Case.

(1st.) When the reservoir empties itself through an orifice.

Velocities.—The ratio between the velocity at the orifice of discharge and that of the water in the reservoir is in the inverse ratio of their sectional areas.

Head.—If H = actual height of water in the reservoir; h — the height due to and generating the velocity of discharge, and A and a are the sectional areas of the reservoir and the orifice.

Then $h = \dfrac{H A^2}{A^2 - m^2 a^2}$.

Discharge.—A reservoir emptying itself through an orifice in a given time would discharge a volume equal to half that due to the head at the commencement, kept constant during the same time. For an example of this applied to locks, see example 4, page xlvi.

Time.—The time in which a prismatic reservoir empties itself is double that in which the same volume would be discharged if the initial head had remained constant.

The time of descent, t, to a given depth, $d = H - h$

$$t = \frac{2\,A}{ma\,\sqrt{2g}}\,(\sqrt{H} - \sqrt{h}),$$

and the quantity discharged in a given time, t,

$$\text{is } Q = A\,(H - h) = \frac{t.m.a\,\sqrt{2g}}{A}\left(\sqrt{H} - \frac{tma\,\sqrt{2g}}{4A}\right)$$

and the mean hydraulic head, H, under which the same quantity would be discharged in the same time is—

$$H_1 = \left(\frac{\sqrt{H} + \sqrt{h}}{2}\right)^2$$

where H and h are the heads at the beginning and end of the time of discharge, the reservoir receiving no supply during that time.

(2nd.) When the basin or reservoir receives a constant supply during the time of discharge.

If q = quantity supplied per second,

t = time in which the surface will descend the depth, $x = H - h$.

$$t = \frac{2\,A}{(ma\sqrt{2g})^2}\left\{ma\sqrt{2g}(\sqrt{H} - \sqrt{h}) + q.hyplog\frac{ma\sqrt{2gH} - q}{ma\sqrt{2gh} - q}\right\}$$

when there is no supply, or $q = 0$, this equation resolves itself into that previously given.

(3rd.) In the case of there being no supply, but the discharge instead of being effected through an orifice is conducted over an overfall—

$$t = \frac{3\,A}{mL\,\sqrt{2g}}\left\{\frac{1}{\sqrt{h}} - \frac{1}{\sqrt{H}}\right\}$$

Non-prismatic reservoirs are extremely difficult to deal with, and the investigation of any special case would be comparatively useless.

Second Case.

When one reservoir empties itself into another.

(1st.) When each of the two reservoirs being exceedingly large practically preserves its own level, the communicating sluice being below the lower surface of water; then if H, h, are the heads—

$$\text{the discharge } Q = ma\sqrt{2g\,(H - h)}.$$

(2nd.) When the upper reservoir being exceedingly large preserves its own level, and the lower reservoir having a definite area (A), receives the supply through a sluice of a section (d), required the time in which the surface of the lower basin will rise to a certain height.

If H, h, be the heads on the lower surface at the beginning and end of the time, t,

$$t = \frac{2A}{ma\sqrt{2g}}\left(\sqrt{H} - \sqrt{h}\right),$$

this formula, like that previously given, is useful for determining the time necessary to fill a lock chamber: when $h = 0$, or the levels become the same, the case is that of canal locks, and the sectional area of the sluice may be determined from this equation.

(3rd.) When neither reservoir receives any supply, and both are limited in size, if the surfaces are originally at different levels, and the communication sluice is opened, the surface of one will rise and the other fall.

If A, B, are the sections of the two vessels,
H, x, the heads at the beginning and end in A,
h, y, the heads at the beginning and end in B,
a = the sectional area of the pipe or sluice,
t = time during which the sluice is open,

$$\text{then } t = \frac{2A\sqrt{B}}{ma.(A+B)\sqrt{2g}}\left\{\sqrt{B\,(H-h)} - \sqrt{(A+B)x - AH - Bh}\right\}$$

[redacted]

if it be required to know the time in which the two faces will be level; in that case, $x = y = \frac{AH + Bh}{A + B}$, l then

$$t = \frac{2AB\sqrt{H - h}}{ma(A + B)\sqrt{2g}}.$$

This formula is convenient for determining the time cupied in bringing the water in the two chambers of a ouble lock to the same level, by means of a sluice of nown dimensions.

12.—THE APPLICATION OF THE WORKING TABLES.

The use of the greater portion of these twelve tables as already been indicated in the foregoing text; they ave for their object not only the reduction of labour n calculating quantities, but also to serve as a check on any calculations of the same nature that may be rapidly made by engineers in dealing with quantities of water. Table I. gives the amount of the force of gravity in different latitudes, and may occasionally be found of use in pendulum experiments, and in such calculations in which the ordinary value of g 32·2 feet per second, generally applied in the hydraulic calculations in the form of $\sqrt{2g} = 8{\cdot}025$, is not sufficiently exact. Tables II., III., and IV., are of use in calculations of water supply from catchment areas, storage, flood discharge, and waterway. Table V. gives some velocities usual under certain circumstances that are occasionally equired, and as to which the memory cannot always be rusted. Table VI. affords a ready means of reducing or onverting gradients and angular slopes into the forms most usually required by hydraulic engineers. Table VII. gives mean velocities of discharge of open channels f all sorts; these have, however, in conformity with modern practice, to be modified by coefficients suited to the particular case under consideration; the various functions of mean velocity can also be easily deduced by the aid of this table. Table VIII. gives discharges of full cylindrical pipes and tubes, and the diameters and heads corresponding to discharges; these also require modification by suitable coefficients in the same way. Table IX. gives velocities of discharge of sluices, the same table serving also for weirs by making a deduction of one-third from the velocities there given. Table X. gives the loss of head due to bends in open channels and in pipes, and the rise of water due to obstructions in open channels and rivers. Table XI. is a table of equivalents, affording the means for a ready conversion of quantities often entering into hydraulic calculation, such as total into continuous quantities; and, especially intended for use in calculations of storage, town supply, and distribution of water in irrigating land. The latter portion of this table consists of conversion tables for English and metrical measures. The greater portion of Table XII. is a collection of all the experimental coefficients necessary in ordinary hydraulic calculations; they have been arranged in this manner in preference to being distributed throughout the tables, in the belief that it permits of greater convenience in reference: part 6 of this table is a small collection of hydraulic memoranda, principally for purposes of conversion, and also of weight and pressure, intended to aid in rapid calculations; and part 7, consisting of useful numbers, having the same object, also serve for readily applying powers and roots to the coefficients that have now become so important a part of all hydraulic calculations. These tables and data have all been calculated and reduced by the author, with the exception of those at pages lviii and lx.

[redacted]

The Appendix to the Working Tables consists of a few miscellaneous tables and data, giving information sometimes required by the engineer in connection with hydraulic works, the last being a table of British-Indian weights and measures; these with two or three exceptions, in which the tables were made by the author, have been taken from the best sources available, and rearranged in a convenient form.

CHAPTER II.

On Field Operations and Gauging.

1. Direct measurement of discharge. 2. Gauging by rectangular overfalls. 3. The measurement of velocities: different appliances and instruments: flumes and gauges. 4. Gauging by means of surface velocities. 5. Gauging canals and streams by loaded tubes. 6. The Mississippi field operations for gauging very large rivers. 7. Field operations in gauging crevasses: and computation of coefficients for special crevasse-discharges. 8. Captain Humphreys' improved system of gauging rivers and canals. 9. General Abbot's mode of determining discharges on any given day. 10. The experiments of d'Arcy and Bazin on the Rigoles de Chazilly et Grosbois. 11. The gauging of great rivers in South America, by J. J. Révy. 12. General remarks on systems of gauging, and conclusions therefrom.

1.—DIRECT MEASUREMENT OF DISCHARGE.

The direct measurement of the discharge of a channel or stream can be obtained by means of gauge-wheels. The channel is widened until the water flows at a moderate depth, less than five feet, over a horizontal and carefully constructed apron which is divided by piers into a number of equal openings. At each of these openings a gauge-wheel is placed, which fits the opening every way within a quarter of an inch. Sheet piling is driven across the head of the apron and along the banks proaching it for some little distance, so as to force the ole of the water of the stream to pass between the piers d drive the wheels. The measurement of the water is mined by the number of revolutions of the wheels, h should be all coupled on to one shaft and be made ecording on a dial-face, and by the dimensions of the ls, or spaces between their blades, as well as by the h of water passing over the apron, which is observed tervals of about five minutes on gauges erected for urpose.

e method of obtaining a discharge by means of e-wheels is expensive and interferes with navigation ell as the passage of the water; it is therefore very y adopted.

.—GAUGING BY RECTANGULAR OVERFALLS.

e water of a canal or stream is made to discharge over a single horizontal dam, or over a series of small falls specially constructed for the purpose. The discharge over overfalls of certain dimensions, and under in circumstances, is known by many series of experis to be correctly expressed by a formula, containing required data and dimensions, known as Francis's la, it is

$$Q = \frac{2}{3}\sqrt{2g} \times m \quad \left[l - 0{\cdot}1\,n\,H\right] H^{\frac{3}{2}}$$

e l = length of weir-sill.
H = head on the weir from still water.
n = number of end contractions.

the weir-sill is of the same length as the breadth of hannel of approach, $n = 0$; if less than it, and there central pier or obstacle, $n = 2$; each central obstacle ing two additional end contractions.

g $\sqrt{2g} = 8{\cdot}025$ and $m = {\cdot}6228$

$$Q = 3{\cdot}33198 \left[l - 0{\cdot}1\,n\,H\right] H^{\frac{3}{2}}$$

s gives results within one per cent. of absolut exactitude. The dimensions in this formula being taken in feet, the discharges will be in cubic feet per second.

The following conditions should be observed in gauging by rectangular overfalls.

1. As regards form of construction, the dam in which the overfall or series of overfalls is placed, should have the sills truly horizontal, and the sides of the overfalls truly vertical: the dam itself should be vertical all along on the up-stream side, but the sills should all be sloped off on the down stream side at an angle of 45° or more with the horizon; all the edges of discharge should be sharp and true, after passing which the water should discharge itself unobstructed.

2. In order to obviate the necessity of allowing for the velocity of approach in the channel, the area of the overfall—*i.e.*, the quantity $l \times H$, must not exceed one-fifth the area of the channel; otherwise an allowance must be made on this account, as given in the paragraph on Weirs, Chapter I.

3. If the velocity of the channel of supply should not be uniform in all parts of its section, arrangements should be made to make it so; this can be done by placing gratings, having unequally distributed apertures, all across the channel, and as far from the overfall as possible, and letting the water pass through them under a small head.

4. In addition to the above it is absolutely necessary that the air under the falling sheet of water should have free communication with the external air.

With regard to dimensions:—

5. Should the overfall not extend to the entire width of the channel of supply, there should be at least a difference at each end equal to the depth on the overfall, so as to produce complete end contraction.

6. When the breadth of the overfall is equal to that of

the stream, and even under all circumstances, the depth on the weir should be less than one-third the height of the barrier.

7. The depth on the weir must be always less than one-third of the length of the sill.

8. The head on the overfall, H, should never be less than ·2 feet; it is better, also, to make it more than ·5 feet and less than 2 feet.

9. The fall from sill to tail-water should not be less than half the depth on the weir in order to ensure a free fall.

The following practical directions suitable to streams and moderate rivers are given as examples, where ordinary care and accuracy is required.

Practical directions.—1st. When the discharge is supposed to be less than 40 cubic feet per second:—

First, according to the above rules, make H greater than ·2 feet; and $H \times l$ less than one-fifth of the channel section; let l be greater than ·3 feet, but less than one-third the width of the channel; and, to ensure a free fall, arrange so that the lower edge of the sill may not be less than half a foot above the tail-race.

Under these conditions the coefficient of discharge to be used will be $m = \cdot 623$, and any error should not be more than one per cent. Obtain the surface velocity (V_s) and the transverse section (S): the approximate discharge will then be $Q_s = V_s \times S$, and assuming a value for l as before mentioned, obtain a value for H by means of the ordinary formula, making use of the approximate discharge for this purpose. H should be from 1 to 3 feet, and should such a value not result, from the application of the previous conditions, use another value for l, so as to secure this condition, as well as to retain the other conditions before mentioned. When this is gained, the orifice may be cut of the required dimensions in one-inch plank well puddled, of which such dams are usually made; and as, in practice, the dimensions are not likely to be very closely adhered to, they should be measured again when the orifice is completed, and applied to the formula before given for this purpose to obtain the velocity of discharge and amount of discharge.

2nd. When the supposed discharge is more than 40 cubic feet per second, but still admits of being dammed:—

Find the approximate discharge from the section and velocity, when the surface of the stream is level with a fixed mark on a post or stone, at from 100 to 200 feet below the intended site of the weir; having previously selected a place where the stream is regular in width and inclination, construct the dam so that the weir-sill may be equal to the full breadth of the channel, square the ends of the opening with planking, and put a gauge at each end, with the zero at the level of the upper edge of the sill of the overfall, which again should be from 1 to 5 feet above the fixed bench-mark.

When the water is up to the mark, read the height on either scale; take their mean, and use it as a value for H in the weir formula before given to obtain the velocity and amount of discharge. If necessary, obtain the surface velocity of approach W, and make allowance for it as before mentioned under the head of weir discharges, as suitable for this case; m being $= \cdot 666$.

3.—THE MEASUREMENT OF VELOCITIES.

There are many cases when it is not advisable to construct a dam or gauge by overfalls, and also cases where the simple calculation of discharge due to the slope of the river, and the terms of its cross section, would not give sufficiently accurate results. Under these circumstances velocity observations must be made, and other data correctly obtained, so as to obtain from them the mean velocity of discharge, which, when multiplied by the sectional area, gives the required discharge.

In all cases where velocity must be observed, it is necessary to choose a straight reach of the river having a tolerably uniform channel section; it is also advantageous that the bank should admit of the measurement of a straight line parallel to the general direction of the channel, and at right angles to the line of intended river section of observation, to serve as a base for triangulation.

For exactitude of result, it is also advantageous where circumstances admit of it to use a flume, should the channel be sufficiently small to admit of it, as this ensures a perfectly regular section of water for a certain distance, and admits of more exactitude in the determination of the sectional area and that of the hydraulic mean radius. A flume is a timber framework covered with carefully jointed plank, forming a complete lining to the bottom and sides of the channel for from 100 to 200 feet in length, having a perfectly equal section throughout; this gives the means of accurately measuring the dimensions of the stream, the whole of the water of which is forced to pass through it by means of sheet piling at its upper entrance. It produces no sensible disturbance in the flow of the water, and does not interfere with the navigation or passage of water. Velocity observations are then made on a measured length along the flume to obtain the mean velocity, which, when multiplied by the section of the flume, gives the required discharge. A long and accurately constructed open aqueduct in perfect order answers all the purposes of a flume. Should, however, no such opportunities for the exact determination of the water section present themselves, it becomes necessary to resort to soundings. These are perhaps best and certainly most rapidly taken by means of a surveyor's 100 feet chain, with a suitably heavy leaden weight attached to one of the handles; some, however, prefer a cord to a marked chain, and consider it better to measure the length of cord with a tape at each sounding.

The determination of the position of each sounding can in narrow reaches of rivers be best made by stretching a rope across the river, and measuring the distances of the sounding points from one bank along the cord. In wide reaches where this is impracticable, the sounding points have to be fixed by angular observation and connected with the base line of triangulation at the moment of sounding either by an observer with a theodolite on the shore, or by one in the boat with a pocket sextant.

The fall of the water surface at all states of the river is one of the data generally required. To determine this, a gauge post is erected, driven into the ground at each sounding section, and the heights of the water shown on them continually recorded so as to show all variations of depth; the connection of level between the two or more gauge posts is made by levelling either from one post to the other, or from both to a fixed bench-mark. In many cases the fall of the water surface is so slight that the ordinary 14-inch level, and staves graduated to hundredths of a foot, of the ordinary surveyor, do not give sufficiently exact results, when a good 18-inch level and staves reading to millimetres might perhaps just answer all purposes.

The gauging of the exact water level, the variations of which are frequently very small though still important, often requires arrangements giving greater precision than that given by a gauge post, or a rod held to the water level. The two instruments employed for arriving at a very exact determination of water level are—1st, Boyden's hook gauge; 2nd, The tube gauge, used by Bazin.

Boyden's hook gauge.—With regard to gauges, it is well known that the capillary attraction of water about any rod placed in it as a gauge for determining the water level will falsify readings, to obviate this the well-known Boyden's hook gauge may be used where extreme precision is necessary. This gauge has a hook at its lower end, which can be raised or lowered by turning a screw; when the point of the hook is even a thousandth part of a foot above the water surface, the water around it is sensibly elevated by the capillary attraction, and obviously distorts the reflection of light from the surface; when the hook is lowered just sufficiently to cause this distortion to disappear, the point of the hook must coincide with the water surface; a true reading, exact within ·001 of a foot, can then be read, by means of a vernier attached to the rod of this gauge which is graduated to hundredths of a foot. As this instrument cannot be used effectively in a current, it is usual to put it in a box in some convenient place which only communicates with the external water by means of a hole, or if the depth at some distance off is the object, by a pipe leading from that place to the hole in the box, any oscillation of the water surface in the box may then be diminished or nearly removed by partially obstructing the hole at will. Should perfect rest not be attainable, a good mean position of the point of the hook may be obtained by adjusting it to a height at which it will be visible above the water surface for half the time. It is sometimes convenient to have the hook made with a small semispherical knob on it, a level-staff can then be held on it for taking a sight with an instrument.

The tube-gauge used by Bazin is, unfortunately, not described in detail, nor are drawings of it given in his "Recherches Hydrauliques." It seems, however, to have been a glass tube having a mouthpiece of only a millimetre in diameter, and that it enabled variations of water level of one millimetre to be easily read; and it is hence extremely probable that it resembled in some respects the velocity gauge-tube of d'Arcy, used for taking velocity measurements, hereafter described. It is, in fact, evident that an instrument on this latter principle, capable of indicating variations of velocity with precision, would also indicate with exactness the moment of the withdrawal from, or submersion of its mouthpiece in, the water, and that this motion could be easily manipulated with a clamping and a tangent screw.

In addition to the above data, it is also advisable to take notes of the nature and quality of the soil of which the bed and banks of the river under consideration are composed, as these have an important effect on the discharge, and to notice what amount of velocity of current is just sufficient to cause erosion in them.

The different modes of measuring velocity are the following:—

Surface velocity is very simply measured by observing the time of transit over a known distance or length of a reach of a river, of any light floating body, a wafer, a ball of wood or cork, or a partly filled bottle.

Mean vertical velocity, or the mean of all the velocities from water surface to the bottom under any point, in a vertical plane, is measured by a rod placed vertically, having a length nearly equal to the depth of the river, loaded at one end, and supported by a float at the upper end. The time of transit of such a rod will then give approximately the mean velocity of the vertical plane in which it moves. These rods or poles are sometimes made hollow and weighted inside, as the painted metal tubes of the Lowell experiments hereafter mentioned, thus obviating the necessity of attaching either floats or weights.

A more convenient mode of observing mean vertical locity consists in lowering from the surface to the bottom, d raising again to the surface any accumulative self-cording current meter. This is an operation requiring treme care; the meter must be sufficiently weighted, d, if necessary, also managed by a cord from an addi- onal boat moored up stream so as to ensure its moving rtically up and down; the lowering and raising of the eter must also be evenly and steadily managed, so that e results may not be falsified.

Mean sectional velocity can be approximately obtained small streams and canals at one operation only by aking a light covered framework nearly the size of the hole cross-section of the stream, and so arranging it by oats and weights that it will assume a vertical position right angles to the thread of the current; its time of ansit can then be noted, and this will be the approximate ean velocity of the section.

Sub-surface velocities. The following are means and ppliances for measuring the force of a current, but most these involve the application of a special coefficient of duction due to the particular appliance, in order to btain the actual velocity in feet per second at any pth :—

1.—By double floats.

A weighted float, consisting of ball, or cube of wood, hollow tin weighted with lead, is sunk to the required pth, being attached by a cord to a small upper float on the rface of the water; the upper float being made of cork, ght wood, or hollow tin, carrying a vertical stick, or wire, r convenience of observation, and the length of cord being adjusted as to prevent the weighted float from sinking wer than the depth at which the current velocity is re- ired. The time of transit of this double float, over a measured or a calculated distance, is observed, and is supposed to represent the velocity of the stream at that depth, independently of any coefficient of reduction.

Another method is to employ a pair of equal hollow balls connected or linked together, the upper one on the surface, and the lower one weighted sufficiently to keep it at the certain depth; the velocity of this double float, as observed on a measured distance, is supposed to be that of the current at half the depth of the lower ball.

2.—By instruments of angular measurement.

A quadrant having a graduated arc has a string attached to its centre, and a ball attached to the string, which is immersed in the stream. The current moving the ball produces an angular change from verticality in the position of the string; the velocity is then equal to the square root of the tangent of this angle multiplied by a coefficient, which is constant for the same ball only.

3.—By the indications of a balance.

A ball is immersed in the stream and attached by a wire to a balance, which registers the pressure. Another very similar method requires a small plate instead of a ball, which is connected with the balance, and which is directly opposed to the current.

The tachometer of Brünings is the best known instrument of this type. It consists of a plate fixed at one end of a horizontal stem, which moves in the socket of a vertical bar, by means of which the instrument either rests on the bottom of the channel or is suspended from above. A cord of fixed length is fastened to the other end of the stem, and, passing under a pulley, is attached to the short arm of a balance, on whose other arm a weight is suspended, being placed in such a position that the equilibrium is established with regard to the force of the current under observation. The position of the weight on the graduated arm of the balance indicates the velocity observed.

4.—By the rotation of a screw.

A light metal screw, similar to that of a ship's patent log, will, when submerged in a current, rotate at a velocity approximate to that of the water in which it is placed. If on the axle of the screw a thread is set turning one or more worm-wheels, the number of revolutions of the worm-wheel will indicate the approximate velocity of the water, from which, by applying a coefficient of reduction applicable to the particular instrument, thus including all allowances for friction and other causes, the true velocity of the current may be obtained. There are several current meters of this type Saxton's, Brewster's, and Révy's, hereafter described, are all modifications of this form. Some of these instruments are not suited to great depths and high velocities; others are made self-recording in such a way as to make allowance in the indicated number of revolutions for the loss of velocity by friction; the latter is a great disadvantage, as it is always practically necessary to test each particular instrument, and make use of a coefficient, however small it may be, in order to obtain accurate results.

The earliest now known instrument of this type is the hydrometric mill of Woltmann, used by him in 1790. The wings on its axle resembled those of a windmill, and were square copper plates, set at an angle of 45°, having their sides ·082 feet and their centres at ·164 feet from the axis of rotation; for small velocities the size and distance of the wings was doubled In great depths this instrument was attached to a bar and lowered from a platform between two boats, and the instrument put in gear or out of gear by means of a cord at any depth. This type of current meter, from its convenience of use in observing velocity at any depth, has been re-invented many times.

5.—Pitot's Tube.

This is a glass tube bent at the lower end; it is sunk to the required depth, and its lower orifice directed against the current: the velocity is deduced from the difference of level between the water in the river, and that in the tube which is forced up by the current. The first improvement of this instrument is that of Dubuat, who gave the orifice of the tube a funnel shape, and closed it by a plate pierced with a small hole, thus considerably reducing the objectionable oscillations of the water in the tube. The next is by Mallet, who terminated the horizontal branch of the tube by a cone, having an opening of 2 millimetres, and made the tube itself of iron with a diameter of 4 centimetres; he also introduced a float and stem which, elevated by the force of the current, indicated heights on a graduated scale. The last improvement was that of d'Arcy, hereafter described.

6.—Grandi's Box.

A box, having a small hole in the side towards the current, is sunk to a certain depth and withdrawn after a certain time; the amount of water in the box indicates the velocity at that depth.

7.—Boileau's Air Float.

A glass tube of fixed length is immersed in a position parallel to the current; the upper end of the tube has a conical mouthpiece fitted to it of any convenient size; the velocity of passage of a globule of air through the tube indicates the velocity of the current.

Some of these modes of measuring velocity have for the present practically fallen into disuse, on account of the very limited range of their applicability; others, on the contrary, have been severally adopted by various hydraulicians in modern times, to the entire exclusion of the rest.

Modes adopted in Modern Practice.

(1.) On the Mississippi Surveys it was determined to use
most simple apparatus, so as to avoid the necessity of
plying any coefficients of reduction to the velocities indi-
ed by them; and double floats were invariably used.
e floats used in the Mississippi Survey were kegs without
or bottom, ballasted with strips of lead, so as to sink
remain upright; they were 9 inches in height,
6 inches in diameter; the surface floats, when of
ht pine, 5·5 × 5·5 × ·5 inches, when of tin, ellipsoids,
es 5 5 and 1·5 inches, the cord one-tenth of an inch in
meter; for observations more than 5 feet below the
rface, the kegs were 12 inches high by 8 inches in
meter, and the cord nearly two-tenths of an inch;
ther the weight of the surface float nor the force of
wind directly affected the observations to any appreci-
le amount.

(2.) On the gauging of the Paranà and La Plata, by
. Révy, the screw current meter, with some alterations
d improvements made by him, was invariably adopted.

For ordinary currents the screw used by Mr. Révy con-
sted of two long thin blades of German silver, having
diameter of 6 inches, and a pitch of 9 inches; the
read of its axis worked on two worm-wheels of 3 inches
diameter, one wheel having 200, and the other 201
eth; each revolution of the screw moved the first wheel
e tooth onwards, the second wheel moving one tooth
wards for each complete revolution of the first wheel;
is allowed of the continuous reading of 40,000 revo-
tions; the two worm-wheels had graduated divisions
ound their circumferences, corresponding to the teeth in
mber and position, which were read off at an index
rough a glass plate covering them. A nut was also used

for clearing the worm-wheels from the thread of the axle of the screw, by means of which the instrument was either put in gear or out of gear by hand; a wire attached also enabled this to be done from above when the instrument was at any depth.

For strong currents, the screw-blades were shorter and stronger, and made of steel. Some of the screws used were only 4 inches in diameter. The divisions on the circumferences of the wheels were found to be too near for convenient reading; 100 and 101 divisions would have been preferred to the existing arrangement of 200 and 201.

These meters were generally used for observing velocities of more than 10 feet per minute, their corrected results being absolutely correct within 1 inch per minute of velocity. They required extreme care and continual watching: the slightest bend or damage to a screw-blade, or any clogging or accidental tightening of a screw being liable to vitiate results.

When in good order, exposure to a gentle breeze is sufficient to keep the instrument revolving;—failing this, cleaning and oiling, or readjusting carefully, is absolutely necessary. In order to keep a check on the observations, a second current meter should always be at hand.

The principal advantage of this description of current meter is the convenience with which it can be worked, and its unvarying utility in observations at any depth of water.

(3.) In the experiments of d'Arcy and Bazin, on the Rigoles of Chazilly and Grosbois, the gauge-tube of d'Arcy, a development of the tube of Pitot, was generally used for taking velocity observations.

Pitot's tube, used in 1732, demonstrated the principle that the difference of water level, *h*, shown by the two tubes, one vertical and the other curved, and directed against the current, was that due to the velocity, and that the latter could be obtained from the former, by making use of the formula $V^2 = 2gh$.

The error in this was caused by the fact that the water in a vertical tube immersed in a current stands lower than the water surface outside; the difference being a quantity dependent on the square of the velocity immediately below the orifice. In addition to this Pitot's tubes had a serious disadvantage in that the oscillation of the water within the tubes, whose orifices were of the same diameter as the tubes themselves, did not allow the difference of level to be correctly observed.

These objections are entirely removed in the improved tube of d'Arcy, which has an orifice 1·5 millimetres in diameter for a tube one centimetre in diameter: in addition to this the lower portions of the tube to which the orifices are attached, have a small diameter, and are made of copper: besides this, two cocks are introduced which add greatly to convenience of manipulation. The lower cock, which can be worked by a wire and lever, enables the orifices to be opened or closed at any moment from above, and thus allows the difference of water levels of the tubes to be read off at leisure, after withdrawing the instrument from the water. The upper cock, after the water in the tubes is drawn up by the breath at an upper orifice, shuts off the air, and enables the difference of water level in the tubes, which is not affected by dilatation or compression of the atmosphere, to be read off above against a scale.

This gauge-tube is described in "Les fontaines publiques de la ville de Dijon, 1856," and drawings of it are given in the "Recherches Hydrauliques" of d'Arcy and Bazin, 1865.

In the latter, the vertical glass tubes are 1·25 m. long, the two small copper tubes below them being enclosed in a copper casing, 0·77 m. long, 0·06 m. broad, and 0·011 m thick, terminating in a sharp wedge-shaped point to reduce the effect of the perturbation of the current. The tubes themselves are affixed to an upright of light boxwood, which is graduated and supplied with a vernier; the whole instrument being attached to an iron standard on which it slides, and to which it can be fixed by screws at any height; a handle turning the instrument directs the orifices in any required direction; and an additional movable wooden arm is used to enable the instrument to rest by means of it on any cross-beam or timber from which the observations are being taken.

In taking an observation with the instrument it is usual to take a mean of three maxima and minima.

The following is the theory of the determination of the coefficient of reduction μ in the formula $V = \mu \sqrt{2gh}$ for any instrument.

If a single curved Pitot tube be placed in a current, first, with its orifice directed against it, and recording a height, h', above the natural water surface; secondly, when directed with it, and recording a loss of level, h'', below that of the natural water surface; and thirdly, when directed at right angles to the current, recording a loss of level h''', then—

$$\frac{V^2}{2g} = m'h'; \quad \frac{V^2}{2g} = m''h''; \quad \frac{V^2}{2g} = m'''h''';$$

and hence—

$$V = \sqrt{\frac{m'm''}{m' + m''}} \sqrt{2g\,(h' + h'')} = \mu \sqrt{2g\,(h' + h'')}$$

$$V = \sqrt{\frac{m'm'''}{m' + m'''}} \sqrt{2g\,(h' + h''')} = \mu' \sqrt{2g\,(h' + h''')}$$

d finding from tables the values of velocities V′ and V″ rresponding to the heights $h' + h''$ and $h' + h'''$; the bove equations become—

$$V = \mu\, V'; \text{ and } V = \mu'\, V'';$$

Hence there is a constant relation between the theoretic height $\frac{V^2}{g}$ due to the velocity of the fillet under consideration and the quantities h', h'', h'''; and the coefficient of reduction can therefore be obtained for any sort or form of orifice by means of a few experiments; also, when once the coefficient of reduction for the instrument is determined, it is unnecessary to make further use of the level of the water, in which the instrument is plunged, in determining velocities.

4.—GAUGING CHANNELS BY MEANS OF SURFACE VELOCITIES ONLY.

The experiments of Messrs. Baldwin and Whistler on discharges of canals of rectangular section are worthy of notice. They obtained discharges on the canals by means of surface velocities and flume measurement, and simultaneously gauged the actual discharges by gauge wheels, with the view of determining practically the relation between surface velocity and mean velocity, for channels of a certain size conveying water at certain velocities.

In one case the flume was 27·22 feet wide, with depths of water from 7·52 to 8·14 feet, having surface velocities from 3·07 to 3·34 feet per second: the observations deduced a mean coefficient of velocity ·857, the extremes being ·838 and ·856.

In the other case, the flume was 29·94 feet wide, with depths of water from 7·67 to 8·85 feet, having surface velocities from 1·91 to 2·77 feet per second; the observations deduced a mean coefficient for the surface velocity of ·814, the extremes being ·797 and ·846.

In other cases, the data of which are not forthcoming, the coefficients of *surface* velocity were ·835, ·830, ·810; *and taking ·829 as the* mean of the five results, it can be favourably compared with De Prony's coefficient ·816, obtained from experiments on wooden troughs 18 inches wide, having depths of water from 2 to 10 inches, and velocities varying from 5· to 4·25 feet per second. Another point which Messrs. Baldwin and De Prony agreed in determining was that their coefficients should be slightly reduced for lower velocities and increased for higher. The result is that the proportion between the surface velocity and the mean velocity of discharge for rectangular channels in plank, and within certain limits of velocity and proportions of cross section, may be said for practical purposes to lie between ·8 and ·85. Under similar local conditions, therefore, the discharge of a canal of rectangular section can be rapidly obtained by a few surface velocity observations, the inclination of the water surface, and the measurement of its section. The more recent experiments, however, of d'Arcy and Bazin show that the above law of velocity does not hold generally; and hence this mode of gauging does not admit of general application.

5. GAUGING CANALS WITH LOADED TUBES; BY FRANCIS.

Under the existing arrangements at Lowell, a daily account is usually kept of the excess of water, if any drawn by each manufacturing company over and above the quantity it is entitled to under its lease. In ordinary times, occasional measurements are sufficiently exact; bu when water is deficient, frequent measurements are made In the latter case, the following is the usual course o proceeding:—

A gauging party, consisting of one or more engineer th assistants, is assigned to each flume where measure ent is necessary; and arrangements are so made that th observations for a single gauging occupy about an hou the intervals during the day being occupied in working out the results, which are immediately communicated to the manufacturers, so that the machinery may be adjusted to the amount of water they are entitled to draw.

The following are the dimensions of the measuring flumes used, and the quantities of water usually gauged in them; the depth of water in the flume generally varying from 6 to 10 feet.

Merrimac	100′ long by	50′ wide,	1500 cub. ft. per sec.
Appleton	150	50	1800 do.
Lowell, M. C.	150	30	500 do.
Middlesex	150	20	200 do.
Prescott	180	66	2000 do.
Boott	100	42	800 do.

The loaded tubes used were cylinders 2 inches in diameter made of tinned plates soldered together, with a piece of lead of the same diameter soldered to the lower end, having sufficient weight to sink the tube nearly to the required depth, thus leaving generally about 4 inches above the water surface. A red-paint mark was made to show the amount of immersion required, leaving a space between the bottom of the tube and the bottom of the canal of 1 foot. The tubes were of thirty-three different lengths, varying from 6 to 10 feet: six of each length were provided for this purpose.

In order to adjust the tube precisely, it was placed in a tank made for the purpose, and small pieces of lead were dropped into the top of the tube, and rested on the mass of soldered lead, and more were added until the tube was sunk to the required depth, when the orifice at the top was closed by a cork. The tubes were allowed to remain floating *for some time* in the tank in order to discover *any leak. If they leaked*, they were taken out and filled

with water to discover the position of the leak, when the
leak was soldered and the tube adjusted again. Th
centres of gravity of the tubes adjusted were 1·78 to 1·9
feet from their bottom ends; and thus being low, th
tubes had a strong tendency to remain vertical.

The tubes were put into the water by an assista
standing on a bridge below the upper end of the flum
a thing requiring a little practice to do well; he sto
with his face up-stream, with the tube in hand, t
loaded end directed downwards, but slightly up-strea
holding it at an angle with the horizon, greater or le
depending upon the velocity of the current. At a sig
he pushed the tube rapidly into the water at the angle
which he previously held it, until the painted work n
the upper end of the tube reached the surface of t
water; he retained his hold of the upper end of the tu
until the current brought it to a vertical position, wh
he abandoned it to the current.

There were three transit timbers placed across
flume, the middle one equidistant from the other t
their up-stream edges vertical, and distinctly graduated
feet from left to right. An assistant stood at each tran
timber to note the transits, the assistant at the mid
transit timber also observing the depth of water in
flume at each transit in a box close to him between
lining planks and the wall of the canal, which commu
cated with the flume by a pipe about 4 feet above
bottom. The box contained a graduated scale, divided
hundredths of a foot, the zero point being at the m
elevation of the bottom part of the flume between
upper and lower transit timbers. The bottom of
flume was very nearly horizontal; the elevations to obt
the mean were taken at 32 points, giving an extre
difference observed of ·027 feet in one case. The co

of the tube, denoted by the distance in feet from the left side of the flume when the tube passes the transit timbers, was also observed and called out by the assistants; the mean course being obtained by adding the distances at the upper and lower transit timbers to twice that at the middle, and dividing the result by four for a mean distance.

The usual method of observing the transits was by means of an assistant carrying a stop watch beating quarter seconds, who walked down and recorded every transit himself; but when greater exactness was required, an electric telegraph made for the purpose was used, by which the transit observers communicated transits to a seated observer from their stations, the times of signals being noted by him to tenths of seconds, according to a marine chronometer placed before him beating half seconds: an assistant was also required to carry back the tubes to the up-stream station. In the usual method before stated, a party of five was sufficient for all purposes. The observations were made at distances apart about 1·5 feet in the cross section, as may be seen in the following gauge record for one set of observations, and the mean velocities of the tubes for these mean distances calculated and plotted on a diagram of section paper having the mean widths in feet of the flume scaled on one side, and the other calculated velocities for those widths scaled on the other; a curve joining these points was then drawn on the diagram, from which the mean velocity for each foot in width of the flume was scaled off and entered in the record; from these the mean velocity due to the total width was obtained 2·4311 feet per second; and since the mean section of waterway between the upper and lower transit timbers was = 41·76 × 8·5294 = 356·188 square feet, the approximate discharge = 2·4311 × 356·188 = 865·929 cubic feet per second.

Gauge record of the quantity of water passing the Boott measuring flume, May 17, 1860, *between* 10.30 *and* 11.30 A.M., *length between transit timbers,* 70 *feet, breadth of flume* 41·76 *feet, length of immersed part of tube* 8·4 *feet.*

Position of tube at starting.	Mean velocity of transit.	Position of tube at upper transit.	Position of tube at lower transit.	Mean position.	Depth of water in flume.
0·0	2·102	·3	·8	·55	8·510
1·5	2·258	1·8	1·6	1·70	8·481
3·	2·318	3·2	2·1	2·65	8·450
4·5	2·473	4·4	4·5	4·45	8·470
6·	2·373	6·2	5·4	5·80	8·445
7·5	2·593	8·2	10·1	9·15	8·438
9·	2·672	9·7	10·4	10·05	8·440
10·5	2·800	10·5	8·8	9·65	8·470
12·	2·713	12·3	10·9	11·60	8·483
13·5	2·778	13·8	15·5	14·65	8·490
15·	2·800	15·2	18·0	16·60	8·500
6·5	2·373	17·0	20·4	18·70	8·498
18·	2·593	18·0	17·8	17·90	8·505
19·5	2·431	19·7	19·0	19·35	8·505
121·	2·280	21·1	20·9	21·00	8·522
22·5	2·201	23·4	29·3	26·35	8·533
24·	2·077	23·7	22·1	22·90	8·510
25·5	2·071	26·5	29·7	28·10	8·495
27·	2·258	27·0	25·2	26·10	8·483
28·5	2·258	28·6	26·5	27·55	8·495
30·	2·414	31·0	34·3	32·65	8·550
31·5	2·500	32·1	30·	31·05	8·630
33·	2·258	32·5	28·1	30·30	8·610
34·5	2·672	34·6	36·7	35·65	8·625
36·	2·431	36 5	35·0	35·75	8·632
37·5	2·456	37·5	35·5	36·50	8·612
39·	2·500	40·1	40·5	40·30	8·578
40·	2·500	39·0	39·6	39·30	8·578
41·	2·397	41·2	40·6	40·90	8·560
41·76	...	...	...	...	...
0·0	2·047	·5	·4	·45	8·471
10·	2·642	9·8	8·7	9·25	8·580
20·	2·174	20·9	19·9	20·40	8·605
30·	2·273	31·5	33·8	32·65	8·635
41·	2·295	41·4	40·6	41·00	8·610
41·76	...	...	...	...	...
				Mean	8·5294

Products of mean velocity and widths.

2·073 × 1 = 2·073
2·193 × 1 = 2·193
&c. 2·284
2·359
2·422
2·478
2·529
2·577
2·623
2·666
2·705
2·744
2·776
2·801
2·811
2·798
2·747
2·648
2·514
2·363
2·242
2·174
2·129
2·090
2·108
2·135
2·160
2 023
2·243
2·286
2·339
2 371
2·413
2·453
2·483
2·513
2·530
2·541
&c. 2·544
2·504 × 1 = 2·500
2·417 × 1 = 2·417
2·264 × ·76 = 1·721
Sum 101·523

Products of each foot of breadth.

$$\text{Mean } \frac{101\cdot523}{41\cdot76} = 2\cdot4311$$

To obtain the true discharge from this approximate ult, an empirical factor, depending on the difference) between the depth of water in the flume, and the epth to which the tube was immersed, divided by the epth of water in the flume, was applied: the expression correction being $1-0{\cdot}116\,(\sqrt{D}-0{\cdot}1)$. The value of is expression for various values of D is given in the tached table at p. 98.

In this case D, the quantity before mentioned,

$$=\frac{8{\cdot}5294-8{\cdot}4000}{8{\cdot}5294}={\cdot}0152\,;$$

hence the true discharge

$$=865{\cdot}929\times\left\{1-{\cdot}116\,(\sqrt{{\cdot}0152}-{\cdot}1)\right\}=863{\cdot}59.$$

Remarks on the application of this method of gauging.

The preceding measurements were made in a flume aced below a quarter bend in the canal, which caused the ocity to be much greater on one side than the other. To viate this, an oblique obstruction was placed near the wer end of the bend, which removed all the trouble in asurement due to the original irregularity; the other maining irregularities may be seen by plotting a diagram the velocities. It is hence advisable in all cases to ualize the velocities on each side of the axis, should ey require it.

In gauging a branch canal it is best to put the flume in near its off-take from the main canal, with its axis nearly rallel to that of the branch canal. Its section may be termined by roughly calculating the expected discharge, making it so as to suit a velocity of from 1 to 3 feet second; its length should not be less than 50 feet, lowing 20 feet above the upper transit timber to enable bes to attain the same velocity as the water, and 5 feet low the lower timber, the transit course of 25 feet, run

Table of correction for Discharges obtained from Tube Velocity observations, being values of the expression $1 - 0{\cdot}116\,(\sqrt{D} - 0{\cdot}1)$ *for different Values of* D (*from the Lowell Experiments*).

D	Correction.	D	Correction.	D	Correction.	D	Correction.	D	Correction.
·000	1·01160	·020	·99520	·040	·98840	·060	·98319	·080	·97879
·001	1·00793	·021	·99479	·041	·98811	·061	·98295	·081	·97859
002	1·00641	·022	·99439	·042	·98783	·062	·98272	·082	·97838
·003	1·00525	·023	·99401	·043	·98755	·063	·98248	·083	·97818
·004	1·04426	·024	·99363	·044	·98727	·064	·98225	·084	·97798
·005	1·00340	·025	·99326	·045	·98699	·065	·98203	·085	·97778
·006	1·00261	·026	·99290	·046	·98672	·066	·98180	·086	·97758
·007	1·00189	·027	·99254	·047	·98645	·067	·98157	·087	·97738
·008	1·00122	·028	·99219	·048	·98619	·068	·98135	·088	·97719
·009	1·00060	·029	·99185	·049	·98592	·069	·98113	·089	·97699
·010	1·00000	·030	·99151	·050	·98566	·070	·98091	·090	·97680
·011	·99943	·031	·99118	·051	·98540	·071	·98069	·091	·97661
·012	·99889	·032	·99085	·052	·98515	·072	·98047	·092	·97641
·013	·99837	·033	·99053	·053	·98489	·073	·98026	·093	·97622
·014	·99787	·034	·99021	·054	·98464	·074	·98004	·094	·97604
·015	·99739	·035	·98990	·055	·98440	·075	·97983	·095	·97585
·016	·99693	·036	·98959	·056	·98415	·076	·97962	·096	·97566
·017	·99648	·037	·98929	·057	·98391	·077	·97941	·097	·97547
·018	·99604	·038	·98899	·058	·98366	·078	·97920	·098	·97529
·019	·99561	·039	·98869	·059	·98342	·079	·97900	·099	·97510
								·100	·97492

over in 7½ or 10 seconds, can be then noticed by a practised observer with a quarter second stop watch.

In gauging rivers by means of loaded tubes, flumes are dispensed with, and marked cords may be substituted for the graduated transit timbers, being supported from the bottom if necessary, so as to be always visible; in large rivers triangulation observations are necessary. The reach should be 50 to 100 feet long, and the bottom irregularities may be removed or filled in to a certain extent beforehand, so as not to interfere with the poles, which should, when immersed, reach to about six inches from the bottom. Boats will be required to convey the poles. As the cross section will be irregular, it will be necessary to divide it into several parts, finding the area and mean velocity of each division, and calculating the corrected discharge of each division separately; the sums of these corrected discharges will then be the true discharge for the river at that spot.

8. FIELD OPERATIONS FOR GAUGING THE MISSISSIPPI RIVER AND TRIBUTARIES, BY CAPTAINS HUMPHREYS AND ABBOTT IN 1858.

Soundings.—The strength of the current, the depth and width of the river, and the floating driftwood, all combined to render an accurate measurement of the dimensions and area of cross sections a difficult operation on the Mississippi. After various experiments, the following system was adopted, by which accurate work was done even in the highest stages of the river. The middle stages were usually selected for this purpose, being preferable to the low stages, during which there would have been exposure to oppressive heat and disease, and more favourable than the high stages, when the difficulties attending accurate measurement were greatest.

Preparatory to making a cross section of the river, whether for general purposes of comparison or for determining a discharge, a base line, varying in length from 400 to 1000 feet, was measured along the bank near the water's edge; an observer with a theodolite was stationed at each extremity of this line. The one directed the telescope of his instrument across the river, so as to command the line on which the soundings were to be made; the other prepared to follow the boat with his telescope, in order to measure its angular distance from the base line when each sounding was taken. The boat, a light six-oared skiff, contained a man provided with a sounding chain, a recorder with a flag, and three oarsmen. The strongest kind of welded jack-chain was employed, to which bits of buckskin were attached at intervals of 5 feet, smaller divisions being measured with a rod in the boat. The sinker, varying from 10 to 20 pounds in weight according to the force of the current, was a leaden bar whose bottom was hollowed out and armed with grease, in order to bring up specimens of the bed of the river. The patent lead was also used for the latter purpose. The boat was rowed some little distance above the proposed section line, and allowed to drift down with the current, the sounding lead being lowered nearly to the bottom. By this precaution, the deflection of the line by the force of the current was prevented. When the first observer, stationed opposite the proposed section line, saw that the boat had nearly reached it, he waved a flag as a signal to take a sounding, and then carefully turned his instrument so as to keep the vertical hair of his telescope upon the point where the chain crossed the gunwale of the boat. The recorder in the boat, seeing the signal, waved his flag to the second engineer to follow *the boat carefully* with his telescope. The man with the

unding chain allowed it to slip rapidly through his
nds until the lead struck the bottom, when he grasped
e chain at the water surface, and instantly rose to a
anding position. This motion was the signal for arrest-
g the movement of each telescope, and recording the
ngles. The recorder in the boat noted the depth of the
ater, and the nature of the bottom soil adhering to the
ad. By the angles measured at the base line, the exact
osition of the sounding, which was never more than a
w feet above or below the proposed section line, was
certained. The process was repeated until soundings
ough had been taken to give an accurate cross section
the river. Careful lines of level were then run up
ch bank from the water surface to points above the level
the highest floods, when such points existed, or to
ther convenient bench-marks. Generally, the triangles
ere computed, and the work plotted before leaving the
ace, in order to fill by additional soundings any gaps
hich might appear on the diagram.

At places where a series of daily velocity observations
s to be made additional precautions were taken, and
o independent sections, 200 feet apart, were sounded
th the greatest care. Soundings, repeated from time
time upon these lines, uniformly showed that no
nsible changes took place in the bed of the river.
e mean of all such sections, when reduced to the same
ge of the river, was accordingly always taken for the
e cross section at the locality. The change in area
oduced by any change of level in water surface could
n be readily computed from the plotted section. To
termine the daily changes of this level, a gauge-rod,
duated to feet and tenths, was observed daily, its
rrectness of adjustment being frequently tested by
mparison with secure bench-marks. An accurate know-

ledge of the area of the cross section on any given day was thus obtained. The tables of soundings for each cross section, which were all numbered, also denoted the distance of the sounding from the base line, the depth of high water during that year, and the nature of the bottom.

Velocity Measurements.—Narrow and straight portions of the river, where the form of its cross section approximated most nearly to that of a canal, where the waters of the highest floods were confined to the channel by natural banks or by levées, and where the river at all stages was free from eddies, were selected for the permanent velocity stations.

The depth and violence of the river rendered the measurement of its velocity, especially below the surface, exceedingly difficult. Of all the methods known for determining this quantity, that by double floats was found to give the best results. The method of conducting these observations was as follows :—Two parallel cross sections of the river having been made as already explained, 200 feet apart, a base line of the same length was laid off upon the bank from one to the other, being of course at right angles to both. This length was sufficient to ensure accuracy without being too great either for observing many floats in a day, or for avoiding local changes in velocity. An observer with a theodolite was stationed at each extremity of the base line. It is evident that, when the telescopes were directed upon the river, with their axes set at right angles to the base line, the vertical cross hairs marked out the lines of sounding upon the water surface, and that the time of passage of a float between these lines was that consumed in passing 200 feet. Also, that if the angular distance of a float from the base line when crossing each line of sounding was measured, its distance in feet from the former could readily be computed, and its path fixed. Upon these principles the observations were conducted. Two skiffs were stationed on the river, one considerably above the upper, and the other below the lower section line, the former being provided with several keg floats. At a signal from the engineer at the upper station, whose telescope was set upon the upper section line, a float was placed in the river. The keg immediately sunk to the depth allowed by its cord, and the whole float moved down toward the lower line. The observer at the lower station followed its motion, keeping the cross hair of his telescope directed constantly upon the flag. At the word "mark" uttered by his companion, when the float crossed the upper line, he recorded the angle shown by his instrument, and then, setting his telescope upon the lower line, watched for the arrival of the float. In the meantime, the observer at the upper station, whose theodolite supported a watch with a large seconds hand, recorded the time of transit of the float across the upper line, and then followed the flag with his telescope. At the word "mark" given by his assistant, when the flag crossed the lower line, he recorded the line and angular distance from the base line. The float was picked up by the lower boat. By this method, the exact point of crossing each section line, and the time of transit, were ascertained. When the velocity was not too great, the time was noted by the engineer at the lower station also, to guard against error. A stop watch was sometimes used. As it was evidently impossible to observe floats daily in all parts of the cross section, the best practical method was found to adopt a uniform depth of 5 feet for all the floats, distribute them equally across the entire river, and afterwards divide the resulting velocities into groups or divisions within which the variation of velocity was but slight; a mean relativ velocity, and a mean relative discharge, for each divisio was then computed, the sum of the latter being a approximate mean discharge of the river, which, when divided by the area of the whole river section, gave a mean relative velocity for the whole river. The resulting discharge, when multiplied by the ratio of the velocity at the assumed depth (in this case 5 feet) to the mean velocity of the whole vertical curve, gave an accurate mean discharge of the river for that place and day.

Computation of Discharge.—A separate plot of each day's velocity measurements was made, in the following manner:—Lines were drawn upon section paper to represent the section lines, the base line, and the water edges The distances from the base line to the points where eac float crossed the section lines were then computed by table of natural tangents, and the points laid down o the plot. Straight lines connecting the two corresponding points indicated the paths of the floats, which wer of course nearly perpendicular to the section lines. Th number of seconds of transit and the depth of the floa was inscribed upon these plotted paths.

The diagram resulting showed that the velocities i different parts of the section increased gradually and quit uniformly with the distance from the banks until th thread of the current was reached, and, since these velocities were found to vary but very slightly for distances o 200 feet apart except in the immediate vicinity of th banks, the diagram of the daily velocity floats was divide by parallel lines 200 feet apart, the first being the bas ine, and the mean of all the velocities of floats in eac ivision taken as the mean relative velocity for that division and recorded. For the shore divisions, unless th *floats happened* to be well distributed through them, th mean relative velocity was assumed to be eight-tenths of that in the outer edge; a rule deduced from a subdivision and study of the velocity when thoroughly measured in these divisions.

For checking and making interpolations for the defective observations of any day in a division, the day's work was also plotted in a curve whose ordinates were the mean velocities of the different divisions, and whose abscissæ were the distances of their middle points from the base line.

The river channel being of a natural form, the sectional areas of all the divisions were unequal, and again the ratios of these areas were not constant for different stages of the river. Each divisional area was therefore multiplied by its mean relative velocity, and the sum of the products was then the mean relative or approximate discharge of the whole section; dividing this discharge by the total area of the whole section, the approximate mean velocity of the river was determined. This computation was made by logarithms, and simplified by the use of a table constructed for the purpose. In order to correct these discharges, which were those due to the velocities five feet below the surface, it was necessary to determine the value of the ratio

$$\frac{\mathrm{U}_m}{\mathrm{U}_5} = \frac{\mathrm{U}_m}{\mathrm{U}_m + \left[\frac{1}{3} + \frac{(\cdot 317 + \cdot 06f)(10r - r^2) - 25}{r^2}\right]\sqrt{bv}}$$

and multiply them by it, thus getting the true discharges, which, when divided by their corresponding areas of cross section, gave the final and correct mean velocity. The numerical values of the above expression or ratio were obtained in the following way, and put into the form of the table given.

The days on which observations were made were g
according to even feet of the approximate mean ve
already computed, it being assumed that the effec
the desired ratio, produced by changes in mean velc
less than one foot, might be neglected. Each grou
then examined in connection with the wind recoı
days were rejected until only calm days, or those on
the wind blew directly across stream, or those on
when combined the wind effects balanced each othe
left. The resulting mean day in each group wa
equivalent to a calm day, so far as wind effect wa
cerned. The following mean quantities were then d
for each mean day by dividing the sum of the qua
by the number of days going to make up the mea
viz., an approximate mean velocity of the river (v), a
reading, and hence a mean radius (r), and mean v
five feet below the surface (U), found by taking a m
the tabulated velocities of all the different divisions.

These values being substituted in the equation,

$$U' = U_{d_{,}} - (\cdot 1856\ v)^{\frac{1}{2}} \left(\frac{d - d}{r}\right)^2$$

putting also $d = 5$, and making $d_{,} = \cdot 317 r$, an
$b = \dfrac{1 \cdot 69}{(D + 1 \cdot 5)^{\frac{1}{2}}} = \cdot 1856$ when D 7 30; the value
was computed and obtained.

Next this value of $U_{d_{,}}$ was introduced into the sam
tion again to obtain new values of U, first for a
$d = 0$, secondly for a value of $d = r$, thus getti
ace and bottom velocities denoted by U_0 and U_r.
ting for these their values in the following eq
ther with those computed for $U_{d_{,}}$ $d_{,}$ and r, the v
was obtained

$$U_m = \frac{2}{3} U_{d_{,}} + \frac{1}{3} U_r + \frac{d_{,}}{r} \left(\frac{1}{3} U_0 - \frac{1}{3} U_r\right)$$

LOCALITY.	mean vel. of river.	down. 4.	down. 3.	down. 2	down. 1	Calm.	up. 1	up. 2	up. 3	up. 4
Columbus	Feet.	...	...	...	...	...	...	...	...	...
	1·6826	·90759	·92250	·93791	·95390	·97040	·98750	1·00521	1·02357	1·04262
	2·4440	·92202	·93519	·94874	·96273	·97737	·99192	1·00721	1·02294	1·03923
	3·6548	·93719	·94826	·95917	·97118	·98302	·99521	1·00767	1·02048	1·03359
	4·5097	·94400	·95407	·96428	·97463	·98546	·99641	1·00760	1·01903	1·03058
	4·3426	·94908	·95829	·96809	·97741	·98723	·99727	1·00689	1·01793	1·02858
	6·6496	·95406	·96261	·97131	·98016	·98918	·99837	1·00773	1·01727	1·02697
	7·4282	·95751	·96550	·97365	·98193	·99035	·99891	1·00762	1·01648	1·02551
	8·3162	·95983	·96747	·97523	·98311	·99112	·99927	1·00756	1·01598	1·02453
Vicksburg	3·6038	·93881	·94854	·95846	·96863	·97895	·98956	1·00037	1·01142	1·02271
	4·4110	·94544	·95458	·96423	·97340	·98310	·99300	1·00307	1·01337	1·02389
	5·5571	·95161	·96017	·96895	·97783	·98693	·99613	1·00557	1·01518	1·02494
	6·7363	·95631	·96440	·97264	·98103	·98952	·99823	1·00706	1·01604	1·02519
	7·0529					·99006				
Natchez	4·6901	·94566	·95501	·96454	·97428	·98420	·99433	1·00466	1·01522	1·02602

N.B.—A calm or wind at right angles to the current = 0; a hurricane = 10.

substituting the resulting value of U_m in the following equation :—

$$\frac{U_m}{U_5} = \frac{U_m}{U_m + \left[\frac{1}{3} + \frac{(\cdot 317 + \cdot 06f)\,(10r - r^2) - 25}{r^2}\right](bv)^{1}}$$

also those already deduced for v and r and b, f alone remained unknown ; by giving f its value successively for each of the various forces and direction of the wind, the table at Page 107 for the stations was computed.

The approximate discharge for each day at each station was multiplied by the ratio in the table most nearly corresponding to its approximate mean velocity to obtain the true discharge, from which the true mean velocity was then obtained.

7.—FIELD OPERATIONS IN GAUGING CREVASSES BY CAPTAINS HUMPHREYS AND ABBOT.

The phenomena observed in the discharge of water through crevasses, or breaks in levées at seasons of high water, were—

1. That the effect of every crevasse, even though as large as 327 feet wide and 15 feet deep, along the line of levée, extends only for a short distance from the bank; in the above instance, it did not affect the line of motion of floating bodies passing 200 feet from the natural bank, or 300 feet from the break in the levée.

2. Between the crevasse and the outer limit of its influence there is always a movement of the water towards the break from all points below and above, which increases towards the break, and rapidly diminishes on reaching the ground in rear of the levée, where it spreads in every direction, but mostly towards the swamps.

3. *There is* a sensible slope along the course of this

In passing the break, whether by a cascade or not, ater is higher in the middle of the opening than at side.

he following was the ordinary method of comput- a discharge. Knowing, from measurements made r the cessation of the flow, the high-water depth of the n crevasse, which was estimated on the line of levée, if material excavation was made there, and on the batture front of the levée, if holes were dug on the line of the break;—the depth on the given day was found by subtracting from this high-water depth the stand of the river below high-water mark—a quantity which was always known either from local information or from a comparison of the nearest river gauges. Taking D to represent this depth, and W, the maximum width of the crevasse after cessation of flow; and knowing from exact information the date of breaking of the levée, and that of the cessation of flow, the width of crevasse of any desired day could be computed; and the required discharge per second was then assumed to be equal to the continued product of this width W, the depth D, and the velocity (v); or $Q = W_{,} \times D \times v$; the velocity when D was less than 4 feet was taken $= 2{\cdot}818 \sqrt{D}$ (Castel's weir formula); and when D was greater than 3 feet, v was taken $= 10 - \frac{17}{D}$; the general formulæ for discharge corresponding to each case being

$$Q = (100 + \overline{n - 4}) \left(\frac{w - 100}{N - 5} \right) D \, (2{\cdot}818 \sqrt{D})$$

$$Q = (100 + \overline{n - 4}) \left(\frac{w_{,} - 100}{N - 5} \right) D \left(10 - \frac{17}{D} \right)$$

here n = number of days of discharge which have pr *ded the given day, and* N = total number of days *scharge*

Coefficient of correction for special cases of crevasses:—

There are cases in which the conditions of the flow of water were considerably modified; such as when the levée was so far distant from the river that the depth at the edge of the natural bank was much less than that at the base of the levée; or when trees, a growth of saplings, or other obstacles existed in front or in rear of the break, both of these causing a diminution of discharge. So when the reported depth of crevasse included that of previously existing excavations on the line of levée, in these cases the resulting calculated discharge would be too high, and it then became necessary to apply in each case a special coefficient of correction. The coefficient for crevasses flowing into the Yazoo bottom was thus determined. The areas of these bottom lands and their watersheds were as follows, in square miles:—

Yazoo bottom	7110	Total.
Yazoo watershed	6740	
St. Francis' bottom	6900	
St. Francis' watershed	3600	34,600
Tennessee and Kentucky bottom	750	
Tennessee and Kentucky watershed	9500	

The yearly rainfall in feet was—

At New Harmony, Indiana	3·92	
At West Salem, Illinois	4·02	
At St. Louis, Missouri	5·18	
Mean downfall at head of region		4·38
At Memphis, downfall for middle of region		4·42
At Jackson, downfall for foot of region		4·99
		feet
Mean for whole region		4·60

Giving total yearly downfall,

$= 34\,600 \times 4{\cdot}6 \times (5280)^2 = 4\,437\,126\,144\,000$ cubic feet.

To obtain the total yearly drainage, the discharge at Columbus, together with that of the Arkansas and White Rivers, was deducted from the discharge at Vicksburg; and from this also a deduction was made of the river during the year between Columbus and Vicksburg being lower by a mean difference of 6·8 feet throughout a mean width of 3300 feet for 589 miles in length; thus getting the drainage

	4 372 572 757 200
Channel drainage ...	69 786 604 800
Total yearly drainage	4 302 786 152 400 cubic ft.

And ratio of drainage to downfall is hence

$$= \frac{4\,302\,786\,152\,400}{4\,437\,126\,144\,000} = \cdot 96 \text{ nearly.}$$

Next, the total rainfall for the Yazoo basin, area 13 850 square miles, for from 1st December, 1857, to 15th July, 1858 = 3·64 feet × 13 850 $(5280)^2$ = 1 405 461 657 600 cubic feet; the mean rainfall 3·64 during that time being determined by register at Memphis, 3·19, and at Jackson, 4·08 feet; applying to this rainfall the coefficient of drainage before determined, the drainage from the Yazoo basin = 1 349 243 191 300 cubic feet.

The area of the Yazoo bottom was dry on the 1st December, 1857, but at high water 15th July, 1858, it had a mean depth of water of 3·08 feet over an area of 6800 square miles; having received between those dates 6800 × (5280) × 3·08 = 583 885 209 600 cubic feet, and the discharge of the channel of the Yazoo, the sole outlet, was measured during this time = 1 408 665 600 000 cubic feet. Hence, 1 992 550 809 600 cubic feet represented the total quantity which, entering the Yazoo basin between those dates, eventually drained off into the Mississippi; and the total amount of overflow from the Mississippi *basin into the* Yazoo basin was 1 992 550 809 60

— 1 349 243 191 300 = 643 307 618 300 cubic feet; this quantity as computed by the uncorrected crevasse formula was—

1 758 153 600 000;

hence the required coefficient of correction for the formula equals the former divided by the latter = nearly $\frac{1}{3}$. This, therefore, holds good for the crevasses in the district for which it is obtained, and the same principle can be applied to any district.

8.—SYSTEM PROPOSED BY HUMPHREYS AND ABBOT FOR GAUGING RIVERS, STREAMS, OR CANALS BY MEANS OF OBSERVED MID-DEPTH VELOCITIES.

The details of field operation to be adopted differ according to the size of the river. 1st. If the river be small and considerable exactness be required, the boat should be anchored at various equidistant stations, the banks being considered two of them, and the station actual mid-depth velocities measured by any of the known methods; the number of stations being sufficient to prevent the velocity of the water between any two of them from varying materially. 2nd. In the case of a large river, if the depth is uniform, sufficient accuracy may be obtained by observing the times of transit of a large number of double floats well distributed across the river section, the kegs being uniformly sunk beneath the surface to a depth equal to half the hydraulic mean radius of the river. Should it happen that the cross section is not sufficiently uniform and symmetrical to admit of this, he site or reach is ill chosen for the purpose. The esults should then be plotted and grouped into divisions of equal width, and the mean result for each division cal*culated, including,* of course, interpolated velocities should *any be missing.*

depth of water in the river should be noted on a ent gauge-post during the observations, or before ter. By this method the results obtained will be first case absolutely, and in the second case nearly, ted by the wind, no matter what its direction or nay be.

method of computing the discharge from these ations will vary according to the accuracy required.

t method.—A close approximate result may be ob- by taking a mean of all the different station or n mid-depth velocities, and applying a coefficient for large, and ·93 for ordinary rivers, to obtain the velocity of the river. In this method there are auses of error which very nearly balance each other, y, the inequality in area of the different divisions, e difference between the mid-depth and mean velo- in any vertical plane, and the above coefficients meet errors. For a rectangular cross section, no coeffi- is required.

ond method.—If greater precision be required, a accurate mean velocity of discharge of the river ay be computed by substituting the grand mean of station mid-depth or division velocities for $U_{\frac{r}{2}}$ in llowing formula,

$$v = \left[\left(1{\cdot}08\, U_{\frac{r}{2}} + 0{\cdot}002b\right)^{\frac{1}{2}} - 0{\cdot}045 b^{\frac{1}{2}} \right]^2$$

formula is deduced by substituting for U_m its value in the general expression,

$$U_{\frac{r}{2}} = U_m + \frac{1}{12}(bv)^{\frac{1}{2}}$$

ducing the resulting equation.

has been already *stated*, when the mean radius *12 feet, b = ·1856,* and under any circumstances

$b = \dfrac{1{\cdot}69}{(r + 1{\cdot}5)^{\frac{1}{2}}}$. The formula therefore gives at once v
the mean velocity of the river; and this simple method i
quite exact in ordinary river sections, though not appli
cable to rectangular sections.

Third method.—Should however a very high degree o
accuracy be required for testing formulæ, or constan
coefficients, an amount of exactitude affected only b
instrumental errors of observation may be secured b
substituting the different observed division mid-dept
velocities successively for $V_{\frac{D}{2}}$ in the formula

$$V_m = V_{\frac{D}{2}} - \frac{1}{12}\,(b\mathrm{v})^{\frac{1}{2}}$$

and the results will be true values of the mean velociti
of the different divisions in terms of $\mathrm{v}^{\frac{1}{2}}$ and know
quantities. The sum of the products of these expressio
by the corresponding division areas, should be plac
equal to the product of v by the total area of the cro
section; and this equation, involving v and $\mathrm{v}^{\frac{1}{2}}$ and know
quantities, will give two positive values of v; the less o
which, corresponding to the actual case when the velocit
is greater at the axis, is the value of the true mean velocit
of the river. This method, though accurate in principle
is probably not so good for ordinary purposes as the previous more simple one, which neglects the latter attemp
at extreme accuracy and involves less observation, and
consequently less instrumental error, as well as less
labour.

—GENERAL ABBOT'S METHOD OF DETERMINING ON ANY GIVEN DAY THE DISCHARGE OF A LARGE RIVER THAT HAS BEEN PREVIOUSLY SURVEYED AND GAUGED.

The previous field operations consist of a survey and merous soundings of a straight and regular portion of e channel between two bench-marks, A and B, fixed manently near the water, whose relative levels are urately known. An accurate plan of the river between se points is necessary, the mean cross section derived m the soundings, and a series of careful gaugings of e river on permanent gauge-posts. It is desirable that e course of the river between A and B should be as aight and regular as possible, in order to eliminate to e utmost the effect of bends, although allowances almost variably must be made on that account. The points A d B should be well chosen, as far apart as practicable, and tant from any eddy, and be placed where the current on e bank flows with equal velocities. The latter coindtion necessary, because water in motion exerts less pressure n when at rest, and if it moves rapidly past one bench- rk, and is nearly stationary at the other, a difference level independent of the motive power of the stream uld vitiate the observations.

On the required day the water surface at each end of e reach, A and B, has to be simultaneously referred by curate levels to the bench-marks, to obtain the difference level of water surface and the gauge depths. Nothing ore is required. A calm day should be selected.

The formula to be used is that given in the paragraph velocities :

$$v = \left[\sqrt{\cdot 0081 b + (225\, r, \sqrt{s})^{\frac{1}{2}}} - \cdot 09 b\right]^{\frac{1}{2}}$$

the terms of which have been already explained, excepting s; in this case s is the sine of the slope of the water surface corrected for bends, and is obtained numerically by subtracting the value of k, due to effect of bends (*vide* Paragraph on Bends) from the total fall between the level stations, and dividing the difference by the total distance between them, measured on the middle line of the channel.

The method of successive approximation must be adopted to find the value of v in this formula. The following formulæ give the value of each variable in the above equation in terms of the others and known quantities; taking $Z = {\cdot}93\ v + {\cdot}167 \sqrt{bv}$ and assuming $p = 1{\cdot}015\,W$, should it not have been measured—

$$s = \left(\frac{Z^2}{195 r_,}\right)^2 \quad a = \frac{(p+W)\,Z^2}{195\sqrt{s}} \quad \text{and } r_, = \frac{a}{p+W}$$

and $p + W = \dfrac{195\,a\sqrt{s}}{Z^2}$?

For small streams.—General Abbot modifies the above formula into the following, where v′ is the value of the first term in the expression for v—

$$v = \left\{\sqrt{{\cdot}0081 b + (225 r_, \sqrt{s})^{\frac{1}{2}}} - {\cdot}09\sqrt{b}\right\}^2 - \frac{2{\cdot}4\sqrt{v'}}{1+p}$$

or putting $M = {\cdot}0081 b$ and $M_, = \dfrac{2{\cdot}4}{1+p}$

$$v = \left\{\sqrt{M + 225 r_, \sqrt{s}} - \sqrt{M}\right\}^2 - M'\sqrt{v'}$$

in which the term involving M′ may be neglected, for streams larger than 50 or 100 feet in cross section; and for large rivers exceeding 12 or 20 feet in mean radius M but not $\sqrt{M}$ may be neglected. The following table facilitates the application of the formula.

r.	M.	√M.	p.	M′.	Log. M′.
1	0·0037	0·0930	5	0·400	9·602060
2	0·0073	0·0855	6	0·343	9·535294
[illegible]	0·0065	0·0803	7	0·300	9·477121
4	0·0058	0·0764	[illegible]	0·267	9·426511
5	0·0054	0·0733	9	0·240	9·380211
6	0·0050	0·0707	10	0·218	9·338456
7	0·0047	0·0685	12	0·185	9·267172
8	0·0044	0·0666	14	0·160	9·204120
9	0·0042	0·0649	16	0·141	9·149219
10	0·0040	0·0634	18	0·126	9·100371
12	0·0037	0·0610	20	0·114	9·056905
14	0·0035	0·0590	22	0·104	9·017033
16	0·0033	0·0573	24	0·096	8·982271
18	0·0031	0·0558	26	0·089	8·949390
20	0·0029	0·0544	28	0·083	8·919078
30	0·0024	0·0494	30	0·078	8·892095
50	0·0019	0·0437	50	0·047	8·672098
100	0·0013	0·0369	100	0·024	8·380211

—THE EXPERIMENTS OF D'ARCY AND BAZIN ON HE RIGOLES DE CHAZILLY AND GROSBOIS IN 1865.

The details of the mode of conducting these experiments, ich were conducted in small channels under various ditions, with the principal object of obtaining coeffici- s of reduction due to various surfaces of bed and banks, cannot fail to be interesting to those intending to gauge channels of any description.

The canal of supply was Bief, No. 57, of the Canal de Bourgogne, from which the water was taken into a receiving chamber through four iron sluices, 1^{m} wide, and being capable of being raised 0·40^{m}, having their sills 0·60^{m} below ordinary water level of the canal. This chamber was 5·40^{m} wide by 14·00^{m} long, having its bottom 0·80^{m} below the entrance sills; the gauge sluices opening from it into the channel of experiment were of brass, twelve in number, each having a section of passage when opened of 0·20^{m} × 0·20^{m}, and having their sills 0·40^{m} above the bottom of the chamber, and 0·40^{m} below the sills of the entrance sluices before mentioned. These orifices resemble those of the type employed by Poncelet and Lesbros, and would, according to them, require a coefficient of reduction of discharge of 0·604, provided that the effect of the velocity of approach be neglected; in this case, however, it augmented the discharge, and an allowance had to be made on that account. The water in the chamber was constantly kept at a level of 0·80^{m} above the centre of the gauge sluices; an appliance for showing the slightest variation of its level being continually watched by a sluice-keeper.

The channel of experiment was 450^{m} long before it commenced to bend towards the river Ouche; it was watertight, and was lined with planks of poplar: its fall for the first 200^{m} was 0·0049 per metre, and for the next 250^{m} was 0·002 per metre up to the bend, after which its fall to the river for the remaining 146^{m} was 0·0084 per metre. The different provisional constructions for employing various inclinations, and sections of different forms, were made in plank within this channel, the spaces being filled with rammed stiff earth. Nails were driven into the bottom of the channel at various points to serve as bench-marks, from ich every variation in depth of water could be obtained h exactitude. Most of the experiments were made by cessively opening the twelve gauge sluices, having one ed section and amount of supply in each case, and thus lve results were obtained for comparison in every experi- nt conducted.

The velocities were principally observed by means of Arcy's gauge-tube, an improvement on that of Pitot; t in some cases also by floats The latter were some- mes simple wafers, and sometimes pieces of wood or cork eighted with lead, 2½ inches in diameter, and 1 inch ick; their times of transit over distances of from 40 to metres were noted by chronometers indicating fifths of conds, and the mean of five or more observations, in hich the float following the course of the axis of the annel was adopted as finally correct.

The following was the mode of determining the measurement of discharge at the off-take.

The coefficient of discharge at the four entrance sluices as determined by closing the lower sluices and noting the me in which the former filled the chamber to a certain eight; in this way the following coefficients were obtained r a head on the sill of from 0·55^{m} to 0·70^{m}, when one ingle sluice was opened at a time.

Sluice raised.	Coefficient.
0·10^{m}	0·645
0·20^{m}	0·639
0·30^{m}	0·631
0·40^{m}	0·621

When the four sluices were opened at once to the full height 0·40^{m}, the coefficient was 0·637, instead of 0·621.

It was hence evident that, in order to obtain a sufficiently

constant discharge, the use of the second set of twelv
sluices became absolutely necessary. The conditions
construction of the latter did not however render the co
traction complete, and hence the coefficients of Poncel
and Lesbros were not applicable to them. In order to ha
effected this, a chamber large enough to entirely annihila
all velocity would have been necessary, the sluices shoul
have been farther apart and their sills should have bee
at least 0·60^m above the [illegible]m of the chamber. It wa
hence necessary also to d[illegible]ine the coefficients of dis
charge for these sluices b[illegible]ct observation.

In June, 1857, experi[illegible] were made with this object
a portion of the channel wa[illegible]osed up, and filled by open
ing one, two, three, &c., up [illegible]twelve sluices at a time, an
the volumes thus discharged in a certain time carefull
measured. The discharges per second were in these cas
from 0·103 to 1·242 c.m.; and when each sluice w
opened separately the discharges varied between 0·10
and 0·1057 c.m., giving coefficients varying from 0·645
0·658. The irregularity of the latter was considered d
to the irregularity of form of the bottom of the portion
channel filled not allowing the exact volume to be calc
lated: hence a mean coefficient of 0·650 was adopted pr
visionally for any number of sluices open at one tim
In 1860, it was determined to obtain this coefficient wi
greater exactitude, and further experiments were made:
the practical details were carefully reinvestigated: t
influence of the variations in depth of the bief or ca
of supply was eventually found to exercise no effect on t
irregularities; the gauge used was supplanted by a gl
tube having a mouthpiece of 1 millimetre in diameter,
means of which variations in depth of water as small a
millimetre could be easily read. The results under th
conditions were thus :—

For a discharge from 1 sluice, the coefficient was	0·633
2 sluices, ,,	0·642
3 ,, ,,	0·646
4 ,, ,,	0·649
5 ,, and upwards to 12	0·650

For a sluice raised only $0{\cdot}10^{\mathrm{m}}$ instead of being fully opened, the coefficient was found to depend on the number of other sluices open, thus:—

When 1 other is opened full, the coefficient for the partly opened one is	0·650
2	0·657
3	0·660
4	0·662
5 and upwards	0·663

The determination of the coefficient for reduction for the gauge-tube.

This was effected by three methods—

1st —By comparing the velocities obtained by means of the tube with the surface velocities shown by floats. The data according to the floats were obtained in channels 2 metres wide, having a discharge furnished by five sluices open at a time: the results gave a coefficient varying from 0·981 to 1·039 as extremes, and 1·006 as the mean of all.

2nd.—By moving the instrument at a known velocity in a mass of still water. The floats and the gauge-tube were drawn by men for a distance of 450 metres, each 50 metres furnishing a set of observations; the obliquities of the course of traction furnished the principal obstacle to arriving at a very exact result. The velocities employed varied from 0·609 to 2·034 metres, giving coefficients of reduction varying from 1·015 to 1·053 as extremes, the general mean of all being 1·034: this was considered fi of various engineers of the French Ponts et Chaussées on the Seine and Sâone.

The second result was the following formula for velocity:

U = the mean velocity of discharge.

V_x = the maximum velocity observed in the section.

$$\frac{V_x}{U} = 1 + 14\sqrt{A}\,; \text{ or } V_x - U = 14\sqrt{RS}$$

or in the form most useful in the cases in which maximum velocities are observed as data for gauging,

$$U = V_x - 14\sqrt{RS}$$

Using values of A from 0·00015 to 0·003 the corresponding values of $\frac{U}{V_x}$ become thus :—

A	$\frac{U}{V_x}$
0·00015	0·854
0·0005	0·762
0·001	0·693
0·002	0·615
0·003	0·566

The above expression, involving terms not included in that of De Prony for the ratio of maximum to mean velocity of discharge, does not admit of comparison with it; but is evidently calculated to supersede it entirely.

The reduction of both of these results to English measures is given in Chapter I.

11.—THE GAUGING OF GREAT RIVERS IN SOUTH AMERICA, BY J. J. RÉVY.

The most recent operations in gauging very large rivers were conducted by J. J. Révy: the account of these is given in Révy's "Hydraulics of Great Rivers" (London, nd includes a description of the method he adopted mining the discharges of the Paranà, La Plata, de las Palmas, and the Uruguay, from which the g brief *résumé* of operations is taken.

ems to have been a work of some time and y to find a reach of the Paranà sufficiently for conducting gauging operations and velocity ments; a hundred miles of the river were searched sfully, but at last a reach straight for many miles nd. Here the river was about a mile in breadth, soundings showed from 5 to 71 feet of water; a xed in the stream did not show a variation of level vater surface of as much as a quarter of an inch in four hours; and the inclination of the water surface nile, was practically nothing.

all observed by levelling for one mile with a 14- el, on equidistant staves placed 300 feet apart, was n ·01 of a foot; it was therefere practically im- under the existing state of the river bank, which adapted for levelling, and with the instruments at carry out levelling operations with any effective as it would have involved ten miles of levelling on ground, and probably required also the use of instruments.

se line of 3000 feet was measured on the low-lying k of the river, with a steel tape of 300 feet; and re set out at right angles at each end of it, to e direction of a river-section-line for soundings; minent points in the neighbourhood and on the nk were triangulated and tied into this base line.

s found that for the surveying and triangulation ther calm weather or clear weather with a gentle was absolutely necessary;—for current observations ys only allowed of operations being carried on.

The soundings on the lines of section were taken by the lead and cord; the length of cord was measured by a tape at each sounding, each of these measurements taking one minute, and the position of each sounding was fixed by angular observation, with a 3-inch pocket sextant giving readings to one minute, on the two flags, one at each end of the base line. The angles were observed in from three to ten seconds each. The number of soundings taken in the section varied with the necessity for them: it was necessary to show, and hence also to find the points in the river bed where there was a change of lateral slope, however many they might be, but in places where this slope was regular and gradual, the soundings were not considered necessary at closer distances than from one-twentieth to one-tenth of the breadth of the river. The section of the Paranà, where its breadth was more than 4800 feet, was sounded in two hours and sixteen minutes, after all the preliminary arrangements, drilling of the men, &c., had been properly carried out.

In plotting the section, the position of each sounding was fixed both by means of the complements of the angles observed at those points, and the calculated distances from the base.

The velocity measurements were made with the screw current meters previously described. As the velocities had sometimes to be observed at great depths, the ordinary method of lowering the meter to its position by sliding it on an iron standard was utterly impracticable, and the following mode was adopted. The current meter was attached to one end of a horizontal iron bar, 9 feet long, 2 inches wide, and half an inch thick, which was suspended by chains passing through rings attached to it from a boat moored over the required spot; in order also to prevent the current from moving the bar from its proper position,

ls from the rings of the bar were also attached to other
boats, one moored 100 yards up stream, the other 100
ls down stream. By these means the current-meter
ld be used with good effect in water up to 100 feet in
th, and in currents up to 5 miles an hour. Four sailors
e necessary in taking current observations in this way.
observations of velocity were generally taken by an
ersion of the current-meter for about five minutes, the
observed by the watch being generally a few seconds
e or less, which were allowed for in the resulting cal-
ted velocity per minute; a second checking observation
also generally made by an immersion of one minute.
instrument was put in or thrown out of gear by means
wire leading from it up to the boat, thus allowing or
venting the revolutions of the screw from recording
mselves on the dial faces at any moment.

In the gaugings carried out, observations of mean ver-
l velocity, giving the mean velocity in any plane from
surface of the water to the bottom, seem to have been
ferred wherever practicable. For these cases, in which it
necessary that the current-meter should be steadily and
nly lowered to near the bottom and raised again to the
face, it was found necessary always to work it from a
tform between two boats, placed 12 feet apart, moored by
r anchors, and to have the two suspending cords marked
every 3 feet with alternately red and white marks, as
des to those lowering and raising them; the cord attached
the down-stream boat was not however considered ne-
ary in this operation, the up-stream cord prevented the
trument from going far out of the vertical direction. In
se operations the instrument was put in gear by hand by
htening a nut on immersion, and put out of gear again
corresponding manner on withdrawal from the water.
aking surface velocity observations, the current-meter

was screwed on to a wooden staff, 3 inches wide and half an inch thick; the revolutions of the screw continuing after withdrawal from the water being at once stopped by hand so as not to vitiate the record on the dial-face.

The determination of the equation of correction for such current-meter was conducted in the following way. It was tested at a low velocity by drawing it through a distance of 189′ 6″ in the still water of a reservoir in a time of 2′ 30″ giving a velocity of 75·9 feet per minute; the average of these trials gave a recorded number of revolutions of 172, or 68·8 per minute: in the same way also it was tested at a high velocity, and showed 176·13 revolutions per minute for a speed of 183·64 feet per minute. The equation of correction being that of a straight line, two points alone are necessary to determine it: on referring these to rectangular co-ordinates on a diagram, and joining them, the true velocity corresponding to any number of revolutions of the instrument could be scaled off from the rectangular co-ordinates to the resulting straight line. Or taking it algebraically, if x and y, x_1 and y_1, be the corresponding pairs of co-ordinates for low and for high velocity,

$$\text{then } y = ax + b, \text{ and } y_1 = ax_1 + b;$$

$$\text{where } a = \frac{y_1 - y}{x_1 - x} = 0{\cdot}9962,$$

$$\text{and } b = \frac{y_1 + y - ax_1 + x}{2} = -6{\cdot}811;$$

$$\text{hence } y = 0{\cdot}9962\,x - 6{\cdot}811,$$

or in the form more useful for obtaining the true velocity, x, from the number of revolutions, y,

$$x = 1{\cdot}00381\,y + 6{\cdot}837.$$

On applying to this equation a value of $y = 0$, we obtain

a result that this particular instrument would cease to
ord revolutions for a velocity of less than 6·837 feet
minute.

Current Observations.—In consequence of the rivers
erved being tidal, and having a variable current, it
s necessary to moor a permanent observatory at a
nvenient point in the deep part of the river on the line
section, and make hourly observations of the current
m it throughout the day and night. The tidal rise
d fall was also registered at every quarter of an hour;
rometric, thermometric, and wind observations were also
orded.

The current observations, both surface, mean, and sub-
face, were taken with Révy's current-meter from a
all boat moored temporarily fore and aft on the line of
tion already sounded, its position in each case being
ermined by angular measurement with a pocket sextant
the extremities of the base line, which fixed it within a
inches. For this work two sailors, two anchors, and
veral hundred yards of line were necessary. The
rrent observations were taken at the surface, and at
pths of 4, 7, 10, 16, and 23 feet, the latter being one
ot above the bottom. The mean current observations
re made three times in each case, and were found to
eck each other within 1·6 foot per minute in observa-
ns giving 80 feet per minute. The time of day of the
rrent observations was always noted, and check observa-
ns were also taken from a fixed level, so that the ob-
rved tidal variation might be applied, and the effect of
e tidal wave—a disturbing cause far greater than that
e to the inclination of the water surface in the cases of
ese rivers—thoroughly investigated.

A convenient mode was adopted for testing the straightness of the reach of the river at the section in which the velocities were observed. The centre of gravity of the river section was found and marked on the drawing, and also the centre of gravity of a section whose depths represented the surface currents in any convenient mode, either feet per minute or per second; the horizontal distance apart of these two centres of gravity indicated the amoun of effect of a bend in the reach at that section. In th Rosario section of the Paranà this was $\frac{1}{278}$ of the widt of the river, and the section was considered favourabl in the Palmas section it was as much as $\frac{1}{2}$ the width o the river, and this was not considered favourable. I cases where a very straight reach is not to be obtaine the position of a section of observation is recommended t be taken at the point of contrary flexure of two reache curving in opposite directions.

The conclusions arrived at by M. Rèvy from his study of the current observations on the La Plata, Paranà, Paranà de las Palmas, and Uruguay, were—

1st. That at a given inclination surface currents are governed by depths alone, and are proportional to the latter. 2nd. That the current at the bottom of a river increases more rapidly than at the surface. 3rd. That for the same surface current the bottom current will be greater with the greater depth. 4th. That the mean current is the actual arithmetic mean between that at the surface and that at the bottom. 5th. That the greatest current is always at the surface, and the smallest at the bottom; and that as the depth increases, or the surface current becomes greater, they become more equal, until in great depths and strong currents they practically become substantially alike.

—GENERAL REMARKS ON SYSTEMS OF GAUGING.

he foregoing brief accounts of the modes adopted various hydraulicians in carrying out field opera- s form a far better guide to the engineer about to ertake the execution of gauging operations than any trary advice, or set of rules, could possibly be; the or may, however, be permitted to make a few remarks onclusion. It is, of course, assumed that the most sable mode of proceeding in one case might not be icable to another, and that the method of gauging ld be suited to the general object, the place, and the umstances. When the object is of an experimental re, having scientific results in view, the experi- talist himself is the best judge of the mode most ed to his object. Most gauging operations, however, for their purpose the determination of the discharges river, or of canals, with as little labour and expense, in as short a time as anything approaching to racy of result will admit; in these cases the amount ccuracy required is that which fixes the mode to be pted.

The most rapid and least accurate mode of deter- ing the discharge of a river or canal at a certain place time is that which dispenses with velocity observations, makes use of a calculated velocity formula as a substi- The dimensions of two parallel sections of a straight h of the channel are measured, the inclination of the r surface between the two is levelled, and the nature quality of the bed and banks are noted; these data ble the discharge to be calculated by the aid of the most ern and most correct formula with a certain amount pproximate truth. The point now to be considered is what amount of exactness may be reasonably expected from the practical application of this method.

The Kutter formula for mean velocity of discharge (fo metres),

$$V = c_{,}\sqrt{RS}\ ;\ \text{where } c_{,} = \frac{z}{1 + \frac{z}{\sqrt{R}}}$$

$$z = 23 + \frac{1}{f} + \frac{\cdot 00155}{S}\ ;\ \text{and } x = f\left(23 + \frac{\cdot 00155}{S}\right)$$

seems theoretically to leave nothing more to be desired except perhaps a simplification of form not attainable in the present state of hydraulic science. It is applicable t channels of all dimensions, from the smallest distribu or rigole to that of the Mississippi; and can be appli to channels of any material, from weed-covered earthe beds to cut stone and carefully planed plank, the data on which it is most carefully based being those of numerous experimentalists. The functions or terms involved are only three, R, S, and f, of which the two former can in most cases be readily and sufficiently exactly observed in practice; the great difficulty, however, lies in the determination of the third function. An examination of the general and the local values of f, given at page lxix. of the Working Tables, will explain this. Among the general values suitable to beds of special construction, from well planed plank to rubble, the value of f ranges from 0·009 to 0·017; and the gradations of roughness or quality of surface are clearly marked by the corresponding values of f, the greatest gap being the difference between 0·013 for ashlar and 0·017 for rubble, a difference that can be easily worked up to in practice without any likelihood of important error. It would hence appear that there would be no difficulty in practice of determining discharges with fair accuracy by means of the above calculated velocity ula for channels constructed in such artificial erials. It is, however, in the cases more usual in tice, namely, in those of canals having earthen beds banks, and in natural river channels, that the values offer so wide a range of choice, that the calculated harge might involve serious error as the result of the tion of an unsuitable coefficient. For earthen canals values of *f* range from 0·020 to 0·035, the gradations which are far from being yet sufficiently definitely ked; and for local values the range is about the e. It would seem, therefore, that in these cases it ld be necessary to determine by velocity measurement discharge of the river or canal under consideration, and ce deduce a value of *f* suitable to it before the above hod could be applied for obtaining its discharge at any or place with sufficient accuracy; or, in other words, actual gauging must be done before this mode of edure can be adopted. In the future we shall ably have the values of this function more definitely down, and we shall then be able to make use of this hod more readily, and with greater confidence in the lts; now we have only the present amount of informa- to guide us, and are hence unavoidably forced into a ain amount of velocity measurement as a means of ectly gauging canals and river channels in earth.

Assuming, therefore, that velocity measurement is ab- tely unavoidable, the question next arises, what is the amount of it necessary in determining a discharge? results of Bazin, determining the relation between the imum velocity in a section and its mean velocity of harge, give the readiest solution of this problem. His ulæ are for metres,

$$\frac{V_x}{U} = 1 + 14\sqrt{A}\text{ ; or } V_x - U = 14\sqrt{RS}$$

where V_x = the maximum velocity, and U = the mean velocity of discharge; and it is evident that by combining with this formula the more modern coefficients of Kutter we can with the aid of only a few observations of maximum velocity, arrive at a mean discharge with rapidity, and a fair amount of accuracy, and be afterwards able to determine a discharge at any time under the same local conditions by means of the ordinary calculated velocity formula and the Kutter coefficient already mentioned, without the use of more velocity observations. The reduction of these equations to English measures is given at page 33, Chapter I.

It is extremely probable that this mode of gauging will be more universally adopted in future, and that a large series of observations will throw more light on the relation of the maximum velocity to the mean velocity of discharge, and enable it to be determined with greater accuracy than is at present possible. Observers are therefore recommended to keep in view in all gaugings conducted on this principle, not only the sectional position of the maximum velocity in a section, which may be confined to a single point either in the middle of the channel at the surface, or at a few feet below it, around which the velocities may diminish in section rather suddenly, or may extend with but little diminution over an important portion of the section, but also the locus of maximum velocity, or its depth below the water surface, which may vary sensibly in a long reach of river; this inclination of the locus, as well as the amount of section of very high velocity, being data that will probably aid eventually in determining the ratio of maximum to mean velocity of discharge with greater precision.

3. The next mode of gauging that seems most applicable
rdinary rivers is one of the modes recommended by
ains Humphreys and Abbot. This, however, involves a

reater amount of velocity observation, and at the same me requires the velocities to be observed at a greater epth, for which all descriptions of current-meters are ot applicable.

The velocities are all observed at a uniform depth equal o half the hydraulic radius of the section, and at equal istances judiciously chosen across the line of section; and he mean of these velocities $U_{\frac{r}{2}}$ is taken;—the mean velocity of discharge, v, is then obtained in the formula,

$$v = \left[\left(1{\cdot}08\, U_{\frac{r}{2}} + {\cdot}002\, b \right)^{\frac{1}{2}} - {\cdot}045 \sqrt{b} \right]^2$$

where $b = \dfrac{1{\cdot}69}{(r + 1{\cdot}5)^{\frac{1}{4}}}$.

This mode should, however, be limited to ordinary and arge rivers; in fact, the application of any of the Mississippi data or formulæ to artificial channels or small streams annot be recommended.

4. The next further attempt at accuracy in river gauging involves a complete investigation of the whole of the velocities in the channel section; the velocity at every point in the cross section should be known and plotted on a diagram, they can then be grouped into divisions of the section by vertical and horizontal lines within which the variation of velocity is not important: a mean velocity for each division is calculated and multiplied by the area of that division to obtain its discharge; the sum of these discharges is the discharge of the whole section. Such detailed observations when carried out on an extended scale involve a large amount of labour, care, and skilled personal superintendence, but at the same time afford results not only valuable as regards the determination of the discharges of the river specially under consideration, but also as records of hydraulic experiment aiding in the progress of science.

CHAPTER III.

PARAGRAPHS ON VARIOUS HYDRAULIC SUBJECTS.

1. On Modules. 2. Modern Irrigation in Italy. 3. The Control of Floods. 4. Towage. 5. On Various Hydrodynamic Formulæ. 6. Irrigation from Wells in India. 7. The Watering of Land. 8. Canal Falls. 9. The Thickness of Pipes. 10. Indian Hydraulic Contrivances.

1.—ON MODULES.

HYDRAULIC engineers not having yet arrived at a perfect module for measuring the amount of water drawn off in an open channel for irrigation or other purposes from an open canal or reservoir, under a varying head of pressure, it is a matter of some interest t examine the older types of design of modules that have been use at various times, and in various countries, before going on to thos of more modern form. The designs being necessarily simple, the will be found perfectly comprehensible by means of descriptic without the aid of drawings or diagrams.

Piedmont appears to have been the birthplace of modules, f although irrigation is essentially Oriental in origin, owing to i extreme reproductive power in hot climates, and though it w introduced into Europe by the Moors, we do not find, either India or in Spain, where portions of these works still exist, anythi approaching to a module. The systems employed in carrying o irrigation almost prove that they had not such a thing at all. I India the practice seems to have been to turn water on to a fie until either the landowner or the turner-on of water was satisfie or perhaps rather until the landowner was satisfied that he cou get no more. No doubt this was the best plan to start with, the object of irrigation was to water the fields sufficiently, and t landowner being the best judge as regards how much water w required for his crop, this mode insured the observation of the proper persons. This plan was, however, open to one very serious objection; when the landowners discovered that an extra amount of water beyond that strictly necessary for the crop was in some cases capable of increasing the amount of produce to a small degree, they would take more water, either by stealth or otherwise; the amount of perpetual squabbling on this subject would then have been very large, had it not been for the fact that in Oriental countries irrigation works were made by rajahs, emperors, or chiefs, whose despotic rule and despotic institutions supplied a very practical limit in such matters—moral or physical force.

In Spain, under Moorish rule, it is probable that this useful substitute for modules was also in vogue; but in the huertas or irrigated lands of Spain in more modern times and under Christian rule, the water being the joint property of several villages that combined to keep the works in order, and legislated for themselves about the distribution of the water, the first great step, the just division of the water on a large scale among the several villages, had to be regularly carried out. The canals being comparatively small, a proportional division was effected by equalizing the size of a certain small number of outlets from the main canal into the subsidiary channels, one village thus taking a fourth or a sixth of the total volume of water passing down the canal.

In Piedmont the conditions were different; the country being hilly, and the water taken from streams and torrents having a considerable fall, water power was extensively used for driving corn mills. It is probable that there were a few water-driven corn mills both in India and in Spain, but there such mills would be public institutions, the miller being a servant of the community, generally living on a fixed income, or yearly pay, given either in kind or in money by all the neighbouring villages using the mill. In Piedmont the mills were the private property of individuals, as they are at the present day in Europe; hence it was there that the first unit of water measurement was arrived at—the amount of water enough to drive a corn mill, which were probably then and there of about the same size and requirements. This amount of water then assumed a technical name, the *ruote d' acqua;* the same thing in Lombardy being called a *rodigine*, in Modena a *macina*, and in the

Pyrenees a *moulan*—the same circumstances in various places leading to the adoption of a similar unit of measurement, which was naturally rather variable. In Piedmont the amount was generally about 12 cubic feet per second, and was supplied by an outlet 19 in. to 20 in. square, the water issuing free from pressure at the surface level. The next step was the introduction of a smaller unit of measurement for purposes of irrigation for discharges under pressure, the Piedmontese *oncia;* which was a rectangular outlet 5·04 in. broad, 6·72 in. high, having a head of water 3·36 in. above the upper edge of the outlet; its discharge was 0·85 cubic feet per second, and this was the immediate parent of the Piedmontese module, and, as far as we know, the ancestor of all modules.

Piedmontese Modules.—These, the most perfect type of which is that of the Sardinian code, were designed or intended to fulfil the following conditions: that the water should issue from the outlet by simple pressure, that this pressure should be maintained practically constant, that the outlet should be made square in a thin plate having vertical sides, that the issuing water should have a free fall, unimpeded by any back-water, and that the water of the canal of supply should rest with its surface free against the thin wall or stone slab in which the outlet was formed. The following is a description of the general type. The water is admitted through a sluice of masonry, having a wooden shutter working vertically, into a chamber in which the water is supposed to lose all its velocity and is kept to a fixed level mark by raising or lowering the shutter; the chamber is of masonry and has its pavement on the same level as the sill of the sluice, the regulating outlet from this chamber being an orifice 7·854 in. square, having its upper edge fixed at 7·854 in. below the fixed water-level mark of the chamber. Its discharge is 2·04 cubic feet per second. If a larger discharge at one spot be required, the breadth of the outlet is doubled or trebled, the other dimensions remaining unaltered. Such are the sole unalterable conditions or data of this module; all its others seem to have varied very greatly; its sill is sometimes at the level of the bed of the canal of supply, sometimes above it, and sometimes below it, in which case a slight masonry incline as made from the bed down to it; the length and breadth of the chamber vary greatly, the former from 15 ft. to 35 ft., its form being circular, oval, or pear-shaped; the side walls splaying outwards sometimes close up to the sluice, sometimes not till near the regulating outlet, the object being to destroy the velocity of the water within the chamber. The lower edge of the regulating outlet is generally, but not always, placed at 9·825 in. above the floor of the chamber. The paved floor of the chamber is in many cases, but not in all, continued at the same level beyond the outlet.

The practical advantages of this type of module consist, therefore, in having a chamber in which the water can be kept to a constant level, and from which the water can issue under a constant head of pressure through a regulating orifice of fixed dimensions.

Milanese Modules.—The *modulo magistrale* of Milan is the most improved type of Lombardian modules, the *modulo* of Cremona and the *quadretto* of Brescia being very inferior to it in design; its principal advantage over the Piedmontese modules being the fixity of dimension of almost all its parts; in other respects it resembles it very much, the principal differences being that the water chamber is always rectangular and covered with slabs, and is hence called the covered chamber, that its flooring has a reverse slope in order to deaden velocity, and that the masonry channel beyond the regulating outlet has fixed dimensions also, a portion of it being called the outer chamber. As to its general arrangements, the sluice of supply has its sill invariably on a level with the bottom of the main canal, which is paved with slabs near it; the breadth of the sluice is the same as that of the regulating or measuring outlet; the sluice gate is worked by lock and level, being fixed and locked at any required height by catch lock and key. As to dimensions, the covered chamber is 20 ft. long, its flooring having a rise of 1·75 in. in that length, and its breadth is 1·64 ft. more than that of the sluice of supply, that is, 82 ft. more on each side; the lower surface of its covering of slabs or planks is fixed at 3·93 in. above the upper edge of the regulating outlet, which is the height to which the water must be kept to secure the fixed discharge. In order to gauge the water in the chamber, a groove is made in the masonry so as to allow a gauge rod to be introduced within at the sill of the sluice, which will read 27·51 in. of water above the sill, when the proper head of pressure exists; should it read more or les sluice gate must be raised or lowered. The outer chamber i in. wider than the measuring or regulating outlet, its total l 17·79 ft.; its side walls, which like those of the covered chaml vertical, have a splay outwards, so that the width at the f end is 11·72 in. greater than at the outlet end, that is to is there equal in width to the covered chamber. To insure fall, the flooring of the outer chamber is 1·96 in. below the edge of the outlet, and has besides a fall of 1·96 in. in its l of 17·72 ft.

The total length of the module is nearly 37·75 ft., but its b is variable, according to the amount of discharge required. tended to discharge a Milanese *oncia magistrale*, the Milanes which varies from 1·21 to 1·64 cubic feet per second accordi different computations, averaging, 1·5 cubic feet per secon measuring outlet is 7·86 in. high and 4·12 in. broad, under stant head of pressure of 3·93 in.; the breadth of the c chamber being 25·54 in., and the breadths of the open ch 13·75 in. and 25·54 in.

It is essential to the effective operation of the regulating that the difference of level between the water in the canal an in the module be at least 7·86 in.; and as the height of wa the latter must be 27·51 in., the depth of water in the canal never be less than 35·37 in. or 3 ft., in order to allow the m to work properly. The following are the relative levels of the of the module, referred to the bottom of the main canal datum :

	Inches.
Water surface in the interior of the module	27·51
Upper edge of the measuring outlet ...	23·58
Upper end of flooring of open chamber ...	13·75
Lower end of the same	11·79

Such is the type of the Milanese modules, the dimensions suitable for a discharge of 1·5 cubic feet per second; unfortun in point of fact, the type has been rarely rigidly adhered to thus its advantages as a universal, or even as a local water sta have been comparatively thrown away in practice. Its use, ever, established a discovery that was at that time very impo

, that larger outlets gave a greater discharge than that due to e proportion of their section for small ones; it was therefore ermined that no single outlet of a module should be made for a charge of more than eight oncia or 12 cubic feet per second; when a greater discharge was required, two or more separate lets were to be used in combination. A gauge post was also nd to be necessary in order to enable the water guardians to ust the sluice accurately.

The principal defect of the Milanese modules is that, owing to rush of water from the canal, it is nearly impracticable to keep onstant head of pressure on the measuring outlet; besides this, d and fine silt vitiate the accuracy of amount of discharge.

Such are the comparatively ancient modules, the Milanese *dulo magistrale* being the most improved one of them. Their e has been very much adhered to in modern times; that of ssrs. Higgin and Higginson on the Henares Canal may be conered as the greatest improvement that can be made on them, thout departing from that type. In this module, the entrance a sluice into a chamber for destroying velocity has been prerved, but the exit is an overfall, and hence more susceptible of ct measurement of discharge; the means applied to deaden the ocity of entrance are again different.

The entrance into the channel through a wall is a passage 6 in. (·6 metre) square, regulated by a well fitting cast-iron door ed by a screw; the chamber is rectangular, 10·37 ft. long, by 0 ft. wide below, 9·20 ft. above, the side walls having a batter 1 in 6. The bottom of the chamber is horizontal and at a el 72 feet below the sill of the entrance sluice. To deaden the ion of the water, a partition of masonry grating is built across chamber at a distance of 4 ft. from the wall, and 5 ft. from the erfall wall of exit, it is 1·37 ft. broad, and has eight slits or verl passages not cross-barred, each slit being 5·4 in. wide. The ter having been deprived of all action by passing through this angement, enters the second portion of the chamber, and then ses over a weir having an iron edge 6·56 ft. (2 metres) long, ed nearly on a level with the top of the entrance sluice, or 2 ft. ve its sill. The discharge required for irrigation being never to eed 176 litres or 6·22 cubic feet per second, the depth on th weir sill will therefore never exceed ·5 ft., the sluice opening 1·97 ft. square.

There are two small side walls having a batter from abo either side of the sluice entrance, these walls projecting in main canal, in order to protect the entrance and prevent silt accumulating there, which otherwise, and perhaps even i case, would have to be dug out occasionally. In order to kee chamber in proper working order, a keeper must be employe a gauge post erected in the canal, with reference to which he l or raises the sluice, and keeps the water in the chamber a at a fixed level.

It is evident that the changes may be rung on this spec module to a great extent without effecting great improvemen increasing the number and altering the positions of the sluice overfalls, and modifying the arrangement for deadening the s of the water. This has been done in many cases without r result; it is hence not worth while to bring forward other exa of this type.

Although some of these are complicated in form, as w much varied in detail, the types are exceedingly simple; th require the occasional attendance of a keeper for adjusting according to the variation of pressure; they are made of brick and masonry, and consist of a series of open passages and co chambers connecting orifices and overfalls. It is quite evident except under special circumstances, such modules are far b the wants of an age that economizes labour, attendance, and s vision wherever possible.

Self-acting Modules.—A module to be of any use now mu the first place be self-acting. Nor, indeed, is this all. A number of self-acting apparatus for regulating the supply or of water have been designed and used, but three-quarters of do not answer all the purposes required of them at present. S are large, some expensive, others involve a large expenditur protective or additional large chambers, others are complicated liable to get out of order, and others involve a great loss of h which, in the case of their application to irrigation canals of s fall, is an insurmountable objection. The worst of them ma id to be those that fail in their main object in producing practical curacy of discharge. With all these objections to deal with, it will ot be necessary to do more than make passing comments on the reater number of them, and the principles involved in their design nd construction.

We will, however, first mention the requirements of a good odule. The primary consideration is that under all ordinary circumstances the discharge may be practically constant and correct, that is, should not be liable to vary more than 5 per cent.; secondly, that it should be very simple in construction and application; thirdly, that it should not be liable to derangement; fourthly, that it be portable, easily applied and removed from any portion of the canal without involving much waste or loss; fifthly, that it should not involve much loss of head, and that it should be able to drain the main canal or basin of supply, down to a level of one foot above its bed, and deliver water if need be as high as within one foot of full level in the canal; sixthly, that it be inexpensive, not costing in England more than about 10*l.*, and more than £5 additional for its attachments, slabs, cisterns, or chambers, and setting it in place in working order.

There are perhaps only three modules yet designed that may be said to fulfil these conditions; these we will for the present term portable modules, and defer dealing with them until after commenting on the others, or ordinary self-acting modules, some of which have advantages or disadvantages worthy of notice, or have attracted special attention in any way.

Until recently, the power of flotation was the sole means adopted in self-acting modules for obtaining an equal discharge under varying heads in the canal or basin of supply. The simplest manner of applying this is perhaps in attaching or fixing the pipe or pipes of supply to the float itself, thus insuring a fixed head of pressure on their entrance, however much the surface level in the supplying basin may vary. So far as this, the modules depending on this principle appear excellent, but unfortunately all of these seem defective on account of other considerations. For instance, in "*the suspended opening,*" where the water enters through two horizontal pipes into the body of the float itself (which is kept submerged to a sufficient depth by weights) and passes out of it through a vertical pipe fixed on to the lower side of it, the vertical pipe has to slide up and down in a species of stuffing-box in a masonry platform below, so as to discharge itself clear of the water in the main canal, and prevent the latter from leaking through into the well below the platform, from which the moduled water alone should be drawn off. This is plainly a contrivance that would be defective for purposes of irrigation; should the vertical pipe not slide easily into the stuffing-box, the power of flotation may be entirely neutralized; should it be too easy there will be leakage, and perhaps to a serious amount; the loss of level is seriously great, the delivery level never being higher than 1 ft. above the bed level of the canal. Modifications of this contrivance, having in view the abolition of the loss of head, have been made by using syphons either erect or inverted, instead of the sliding vertical pipe. They certainly attain that object, but introduce new defects sufficient to render them less useful for purposes of irrigation than the original suspended opening; they are expensive, and difficult to manage, the action of the syphons is liable to be stopped by accumulation of air, and their discharge is not only practically low in comparison with their theoretical calculated discharge, but also is variable, as they are very liable to foul; their adjuncts, chambers around and attached, are expensive. The vertical pipe arrangement of the suspended opening is the principle on which many water-meters, used by water companies for discharging water in large quantities, have been constructed.

The same principle has been adapted to purposes of irrigation in the module of M. Monricher, on the Marseilles Canal, constructed between 1839 and 1850: it is intended to supply irrigation channels having discharges of from 1·06 to 4·24 cubic ft. (30 to 120 litres) per second as a constant supply. The details of construction are as follows: A masonry reservoir 11·15 ft. by 14·76 ft., having its bottom at a level approximately 3 ft. below the bottom of the canal, is connected with it by a rectangular masonry passage having a horizontal masonry covering at the level of low water surface in the canal; a transverse masonry wall stops the action of the water, which enters the reservoir afterwards by two passages, one on either side, the wall and passages taking up a portion of the reservoir space. Beyond two pairs of grooves for putting in stop-planks for shutting off the water entirely during repair, there is no sluice or check to the free flow of the water. In the centre e rectangular reservoir is a cylinder of masonry, having an nal diameter of 2·30 ft., being 1·00 ft. thick, the bottom of it g approximately 2·00 ft. below the bottom of the reservoir, and p edge about 2·00 ft. below low water canal surface. An iron der is made to fit the internal masonry closely, and to slide up down it, and to hang by a rod and adjusting screw to a wooden supported by two wooden floats placed clear of the masonry, of which is 1·64 ft. deep, 1·31 ft. broad, and 5·24 ft. long. e are also two vertical bars in the reservoir outside the floats, d down which the bar slides on rings. The adjusting screw es the iron cylinder, which is about 5·8 ft. long, to be placed at its upper edge may be set at any depth below the water e, so as to produce any required discharge. This, when once and checked, is never altered. The whole is enclosed in a building.

e water of the reservoir therefore enters the iron cylinder , and flows out below; the lower water being divided from the of the reservoir above by masonry partitions, it rises through asonry passage thus made into the masonry water-course or tion channel, the bottom of which is not more than ·75 ft. that of the bed of the main canal; the channel section is ft. by 1·31 ft., having a small enlargement 3·28 ft. square at mmencement of the channel. Plans and details of the module described are given in Moncrieff's "Irrigation in Southern pe."

this module, therefore, the section of outlet, viz., that of the ylinder, is constant; the edge of the cylinder rises and falls tation; the loss of level is as small as can be conveniently ed in modules of this principle of design, and if the cylinder , without much care or superintendence, be made to work well masonry without leakage or friction to any detrimental extent, ted by the engineers of the Marseilles canal, the amount of racy of discharge cannot be great. It would doubtless be an vement were some arrangement applied to this module for ting silt from entering the reservoir, which must be liable to ere with the working of the cylinder, and produce a greater orating effect *in this module* than in many others. The masonry portion of the module would require good workmanshi and the putting together of the whole in good working order c siderable care. It is, therefore, rather expensive, and certainly h not the element of portability.

The Suspended plug is like the suspended opening, a princ that has been adopted for modules and applied in a very large vari of ways, some of which involve complexity of parts and detai Its main principle is probably slightly more modern than that of t latter: both are decidedly old, but as these old contrivances a perpetually being re-invented, a brief description of their princip may be of use to some, while comments on them may deter oth from wasting their energies on an idea that appears to have be fully worked out.

The simplest case of the suspended plug is this. A circu orifice is fixed in a floor at the level of the bed or bottom of t canal or reservoir, and a plug of varying section is suspended i it, being attached to a float that rises and falls with the surface the water; the annular water passage thus left open is made t discharge equal quantities under varying heads by proportioni the section of the plug throughout its length; the area of th annular opening being in inverse proportion to the velocity of di charge. To insure a free fall there is a well below the floor int which the water falls to a depth equal to that of the depth of the flo from high-water level of the canal. The depth of the float and it attachment to the plug prevent its acting at a depth of water of le than one foot in the canal. These two points, which are serio objections to the adoption of this module on irrigation canals, hav been much modified in the more complicated modules construct on this principle, which will hereafter be mentioned. As to th plug itself, it is either a conoid hung in a circular orifice, or a flat sided conoid of equal thickness in one direction hung in an orific which is rectangular laterally and of circular curvature transversely in the latter case a fixed area is left open on the flat sides of th plug which has to be allowed for in the calculations for the sectio of the plug. The diameter of the plug in the case of the conoi is obtained by calculating the areas required to pass the require *discharge for* various heads of water, as, from 1 to 10 ft. for ever hes, and deducting these from the fixed area of the orifice, ainders are then the areas of the circular sections of the those depths from which the diameters are obtained. The oid can be made of the same lateral section for all dis- the thickness of the flat sides being increased in direct on.

ollowing is an example of a module designed on the sus- plug principle, and is perhaps the simplest application of it l practice. It was designed by Don Juan de Ribera, pro- the Lozoya canal, or canal of Isabella Segunda, and is that canal with good effect.

so arranged that the size of the outlet diminishes when the water increases. The module itself is a long tapering plug. ·524 ft. in diameter at its lower end, and is attached rcular brass float above, which floats freely in the water asonry well 3·38 ft. by 3·94 ft. square and 4·16 ft. deep; bottom of this well, which is on a level with the bottom of canal and the rectangular masonry passage connecting a circular orifice 1·56 ft. in diameter, within which the d of the module is made to work vertically, the plug and ing of bronze to prevent rust. Below this well again cond one, into which the water falls after having passed the ring between the orifice and the plug. The entrance ectangular passage leading from the canal, which is only ft. long, is protected from silt by an iron grating, covered in at the top by slabs to the full level in the he well is also covered in by a locked iron trap-door. module friction is reduced to a minimum; the module eely from the centre of the float, and can be slightly raised ed in order to diminish or increase the discharge passing the ring or space between the edge of the orifice and the t when a constant discharge is required it is finally adjusted, and then entirely left alone. The float is about diameter, having a thickness in the middle of about ·9 ft., he edges of ·6 ft.

module discharges one cubic metre (35·3166 cubic feet) per d is hence styled an horametre, the discharge being ·2777 ·0098 *cubic feet per second.* The curve of the module or

bronze plug is such, that the roots of the vertical abscissæ vary i
versely as the differences between the squares of the radius of t
orifice and of the horizontal co-ordinate. Hence, if the requi
discharge is given with a head of water of one metre, when t
diameters of the orifice and plug are respectively ·20 and ·1
metres, then, if the head of water be reduced to ·81 metres, t
diameter of the plug at the level of the orifice must be ·16
metres, as

$$\sqrt{1} : \sqrt{\cdot 81} :: (\cdot 20)^2 - (\cdot 1610)^2 : (\cdot 20)^2 - (\cdot 1653)^2.$$

The lengths corresponding to the different diameters of the ta
of the plug will, for a constant diameter of orifice of ·20, be
follows :—

Depths from water surface	·10	·12	·16	·41	·77
Diameters of plug	·00	·0585	·0912	·1211	·1
Depths from water surface	1·26	1·90	2·71	3·71	
Diameters of plug	·1480	·1554	·1610	·1653.	

The principle being that the velocity of discharge through an ori
varies with the square root of the head of water; thus, taking R r
to represent the radii of the orifice and plug respectively, the discharge per second

$$Q = c\pi(R^2 - r^2)\sqrt{2gH},$$

H being the head of water, the value of the experimental coefficient, c, being for this case deduced, from a series of experiments of Do
Juan de Ribera, to be ·63, in accordance with similar results obtained in ordinary practice in parallel cases. This is probably the module in most perfect accordance with theory yet designed; it is, however, of small dimensions, and hence likely to be much affected by even the very small proportion of silt that would pass through the grating. Its principal defect is, that the loss of level necessarily involved in it in order to obtain a free fall would render it inapplicable in a very great number of cases, where even a few inches of fall are of extreme importance.

The modifications of this type of module consist in putting the float in a separate chamber, which thus becomes a silt trap, and relieves the orifice from being affected by silt, the connection *between the* float and the cone being either a chain passing over nners or a lever: in these cases the plug is reversed, having ader end upwards; the friction involved affects the working module and its accuracy of discharge, and, in the case of the lengths of the arms modify the quantities employed calculations of sections of discharge. In some cases the form lower well assumes various forms, having for their object duction of the loss of level existing in the more simple type. xtremely doubtful whether any of these modifications can be ered advantageous on the whole.

ing and Falling Shutters.—Contrivances of this type are ge- suited for large quantities of water where great accuracy is quired. The falling shutter, as used on canals in England tland, is an oblique shutter hinged below, and raised or low- front of an opening in the side of the canal by two floats in es, the water passing over the upper edge of the shutter in a ly uniform volume. The rising shutter is a vertical shutter t of an opening in the side of and down to the bottom of the it is raised or lowered by means of a float attached to it by passing over a runner, the float being in a separate chamber, ving trunnions and friction rollers running in curved grooves esses on each side of the chamber; these curves require very te construction in order that the discharges may not vary different heads. Shutters of this description having pres- n one side only are very liable to stick, and get out of order; re hence very inferior in practice, although new ones under able conditions can be made to work very accurately.

above three types comprise the whole of the non-portable ting modules that have been much used in practice to good

table Self-acting Modules.—In this class we comprise such les as could be removed or replaced without much difficulty or There are three such modules that have attracted attention, h there are probably others not so well known.

first is that of Lieutenant Carroll, of the Royal Engineers: nciple is exactly that of the well-known draught regulator: essure of the water is made to regulate the opening in the one case in the same way as an increased draught of air is m to partially close the opening in the other; and the applicatio the principle is excellent for the intended purpose—it can be m almost entirely of iron, is simple, effective, and admits of rem without causing much loss or expense. Drawings of this mod are given in the Rurkhi Professional Papers.

The second is a modification of the hydraulic lift regula invented by the late Mr. Appold, used to regulate the descent hydraulic passenger lifts under a variable load; it has been app to its new object by Mr. W. Anderson of the firm of Eastons a Anderson, and in some respects resembles the mode of Lieute Carroll: the velocity through the pipe of discharge is, however, this case made to move a suspended plate of curved form, in fr of an opening also fixed inside the pipe, and the opening is the fore reduced by increase of velocity.

In December 1866 some experiments were made with a 6-i Appold regulator at the request of Col. Smith, engineer to Madras Irrigation Company, and of Mr. Clark, hydraulic engin to the Municipality of Calcutta.

In one experiment, in which the regulator was used to disch water from a tank 7′ 7″ square internally during 13 minutes; t surface of the water in the tank sank as follows, in one minute i tervals: $3''\frac{5}{16}$, $3\frac{1}{4}$, $3\frac{5}{16}$, $3\frac{1}{4}$, 3, $3\frac{3}{16}$, $3\frac{1}{8}$, 3, $3\frac{1}{16}$, 3, $3\frac{1}{16}$, $3\frac{1}{8}$, 3″; the total quantity discharged in 13 minutes was

$$= 7' \, 7'' \times 7' \, 7'' \times 3' \, 5\tfrac{1}{8}'' = 197{\cdot}22 \text{ cubic feet,}$$

or about 15 cubic feet per minute.

In the second experiment, the surface of the water in the ta sank as follows, in one minute intervals: $3''\frac{5}{16}$, $3\frac{5}{16}$, $3\frac{3}{8}$, $3\frac{1}{2}$, 3 $3\frac{11}{32}$, $3\frac{3}{8}$, $3\frac{3}{8}$, $3\frac{3}{8}$, 3, $3\frac{5}{16}$, $3\frac{1}{4}$, $3\frac{3}{16}$, $3\frac{1}{4}$, $3\frac{1}{8}$, $3\frac{5}{16}$, $3\frac{9}{16}$, $3\frac{11}{16}$, $3''\frac{5}{8}$; the total quantity discharged in 20 minutes was

$$= 7' \, 7'' \times 7' \, 7'' \times 5' \, 8'' = 323 \text{ cubic feet,}$$

or about 16·13 cubic feet per minute.

In the latter case the heads at the beginning and the end the discharge over the centre of the pipe were 22·8 feet and 12· feet.

In each case the same regulator or module was used; i aperture on the delivery side was 5″$\frac{17}{32}$ high, and 3″$\frac{11}{16}$ or a section of 20″·35; the swinger was 3″$\frac{7}{8}$ wide, nearly ng at top and bottom; the case 5¼ wide, and the area for passage 8$\frac{9}{16}$″ × 1$\frac{3}{8}$″ = 11″·77 in section.

of these Appold's modules are believed to be in use on the addra canals of the Madras Irrigation Company. From the nience of form that this module possesses, being self-contained, xternally a simple iron tube, with an enlargement like a box middle of it, that admits of being attached or detached from fice very rapidly, it would appear to be far preferable to that ut. Carroll, and less liable to damage in transit.

third portable self-acting module is the design of the author work, and is named the Equilibrium Module. It consists first place of a box or chamber, having an entrance and an rifice, and one or two air holes above; within this box is the eading from the entrance orifice for a short length horizon- and then turning vertically upwards; this is terminated by a end, but has two or four slits or narrow vertical openings in des, through which the water passes when the module is open orking. There is at all times enough water within the cham- rise above the level of these openings, and to work a float them; this float, working vertically, raises or lowers the cap lides over the head of the pipe, and gradually opens or closes lits with the variation of the level of water in the chamber; must of course be below the low water surface of the canal nk of supply. The form of construction adopted reduces to a mum the depth from the water level within the chamber openings, which discharge above the sliding collar, and causes the loss of head to be unimportant.

is is also a small module, possibly only a quarter larger than Appold module before mentioned, and equally convenient egards portability; it is simple in design, being actually more than one of the old types of equilibrium steam applied as a module in a chamber under pressure: it however, be made of any size, the adjustment of the of the orifices of entrance, of exit, and of the slit- ngs being the only important points of variation. It also, for rough purposes, be made generally of stone-ware, and the pipe would then be square in section and have onl two slits, the other two sides forming part of the box. Thi module slightly resembles the old cylinder sluice, which is al a modification of a double beat steam valve; the latter, howeve is not so simple, being far more liable to choke or get out order, one of its valves working within the pipe, and it i therefore not so effective in constant use as any of the thr already mentioned are likely to be.

2. MODERN IRRIGATION IN ITALY.

The persistent increase of prices of the necessaries of lif in all civilized countries has, during the last half-century, been mitigated by improved communications—the railway the steamer—with countries less civilized, but more capable production. That a further and wider extension of such communications will continue to produce a mitigating effect we have little doubt; but afterwards, what have we to look to? Many of the expensive requirements of civilized existence admit of substitutes. For coal we may substitute peat fuel or petroleum; for fabrics hitherto necessary, others less expensive, obtained from plants and grasses hitherto neglected, but now forced by research and skill into the service of man; but, as regards our more urgent wants—bread and meat—there is not now the slightest probability of any substitute being found that could materially relieve the demand for them. We may substitute one kind of meat for another, or one kind of corn for another, as bacon for beef, and maize or millet for wheat and barley: but this is merely economizing by reduction; so we may safely assume that increasing the production of grain and grass throughout the world is the principal mainstay in the future.

In highly civilized countries, where there is comparatively little land fit for culture not already under cultivation, and where high farming has already been adopted to obtain increased produce, it may be assumed that the best results have been nearly reached; it is therefore to less civilized and more distant countries all over the world that we must look for increased produce mainly, and, in the first instance, by increasing and improving the culturable area.

Of all means of increasing agricultural produce, irrigation stands justly at the head, increasing the yield of the very best lands, rendering inferior lands capable of yielding crops of a superior kind, and apparently nearly useless lands, such as much of the sandy arid plains of India, of yielding good crops of different descriptions; the increased yield obtained by these means supporting men and cattle, and causing, through the manure derived, an additional source of increase. The development, therefore, of irrigation everywhere, its means and methods, its economical application, and the investigation of its results under different conditions, become subjects of interest, not only to the professional hydraulic engineer, but of vital importance and consequent interest to every being existing on the face of the earth. Leaving the history and archæology of irrigation for the consideration of the engineer devoted to such subjects, contemporaneous irrigation has besides a still further interest for the capitalist, everything pointing to the probability that, in and for the future, capital will be largely applied to works of irrigation; the countries where irrigation is likely to be most productive being generally incapable for the present of using capital of their own, and the communications on which capital has been so largely utilized having been so far developed as to set free a large capital for other purposes.

The most interesting irrigation, therefore, will not only be contemporaneous, but that which is most instructive as regards results. The project for the irrigation of a tract of land in Lombardy by the waters of the Lago Maggiore, being carried out in 1872 by a small company of local shareholders, under a concession granted by the Italian Government to its engineers, Eugenio Villoresi and Luis Meraviglia, seems to satisfy these conditions in every respect. The works are not large, it is true; but it does not partake of the nature of an experiment, having an element of stability in it, firstly, from being carried out in a country more or less permanently irrigated since the Middle Ages, and hence instructive as regards the development of principles, and, secondly, from being the result of local effort forcing itself forward, and succeeding by acting with the wishes of the population, independently of foreign aid.

The comparative smallness of the project, again, has its advantages, in point of interest, from allowing a perfect development within itself, and is thus more truly instructive in showing what might be done on a large scale with large capital, and by the application of the more extended principles not yet adopted in Italy, but already plainly indicated in the large Indian works of irrigation. Some of the details of the scheme and of the intended results will be interesting in comparison with similar data for Spain and India.

The following information with regard to the Lago Maggiore irrigation project, and local matters in connection with it, was obtained during a visit in 1872, from or through the Director of the College of Engineers, the Director of the School of Agriculture, Signor Cantoni, and principally from Signor Villoresi himself.

The tract of land to be watered from the Lago Maggiore is almost entirely in the Milanese province, and is bounded by the Naviglio della Martesana and the branches of the Naviglio Grande; its area is 216 234 acres and its population 459 166. It is peculiarly dry, from causes that have not yet been explained; the inhabitants either collect rain-water, or draw from wells 40 to 100 feet deep, and scanty in the best seasons, or obtain from the pools of the River Olona the water for their domestic wants. The springs or sources of the Olona are now probably less productive than they were, and as its supply is cut off above, for irrigation purposes for an adjoining canal, it is nearly dry in the region under consideration, the eight or nine torrents running into it being of little value. There are also eight torrents running towards the river Lambro, towards the east; but the whole of these, including the springs and the Olona, are not sufficient for the irrigation of 1500 acres of ordinary cultivation according to the usual Italian practice. The tract of land has a generally uniform fall from west to east, and from north to south, of ·75 and ·20 per 100; the soil is alluvial, and classified into four gradations of mixture of sand and clay, covered with a vegetable stratum 7 to 14 feet thick, and occasionally more; the most sandy portions admit of being irrigated with good effect, and generally consist of pasture land; on the whole of the rest, however, crops are grown independently of aid, excepting the portions covered with heather and woods, which, from continual cutting, have nearly disappeared. For the crops, the rotation in vogue is biennial; in the first year a first crop of wheat or rye, followed by autumnal maize or millet of some sort, in the second year spring maize. Very small quantities of vegetables, flax, hemp, and ravizzo (colza) are sometimes grown; in some parts of the wheat-growing land trefoil is sown among the wheat in the spring, so as to obtain a first cutting from it in the autumn, and a second in the following spring, but this is very rarely successful for want of sufficient moisture: over a larger portion vines and mulberry trees are planted; in all cases the coloni (rentiers, tenants) paying the proprietor in kind, or taking, in the last case, part of the produce in payment for their labours.

Now, even in its unirrigated state, this is certainly not an unproductive region; there is no mention made of deficiency of crop, and the population is thirteen to an acre, although a certain proportion of the land is scrub, heather, and woodland; and yet the inhabitants have set to work energetically to irrigate and increase the produce. Over how large an area of the world is there not land yielding not one half of this without the slightest efforts being made to introduce irrigation! What millions of acres not yielding a quarter of this, in India, are allowed to remain unirrigated, or, as the contemplative Anglo-Indians in charge would say, uninterfered with!

The introduction of irrigation would, under these as well as almost any circumstances, involve an agronomic change, and a different succession and rotation of crop, to which in this, as in all cases, a certain proportion of the cultivators and proprietors are strongly opposed, although they must, from their close vicinity to other irrigated lands, be fully aware of the advantages of irrigation. It seems difficult to fully account for this feeling so often shown in similar cases. Water has to be paid for no doubt; but there is more produced wherewith to pay. Is it the timidity of entering on matters on a larger scale, and want of self-reliance in adapting themselves to a new system; or is it that unreasoning obstinacy so generally ascribed to agriculturists? Whatever it may be, the difficulty in this case seems to be partly met by the School of Agriculture, established at Milan, from which more extended ideas on such subjects are disseminated through lectures and ready information within the means of all.

The first agronomic change proposed is the reduction of the whole of the scrub, heather, and woodland into culturable soil; the second, a great reduction of the vine-growing area for the purposes of cultivating corn, the latter change being justified by the fact that the greater part of the wine produced in this region is of very inferior commercial value. The wisdom, however, of this latter change seems open to objection; as a better cultivation of the vine-growing area, combined with winter floodings, could hardly fail to produce a larger amount of wine. Assuming, however, that this change is desirable (and several landed proprietors have adopted it), it will, when general, reduce three-quarters of the vine-bearing area into cultivated or pasture land. The third agronomic change is that of the formerly cultivated land; the biennial rotation having, under irrigation, to give way to a more comprehensive arrangement. A typical rotation has been laid down which is quinquennial, according to the following table:—

of Crop.

No. of parts.	First Year.		Second Year.		Third Year.		Fourth Year.		Fifth Year.	
	1st crop.	2nd crop.	1st crop.	2nd crop.	1st crop.	2nd crop.	1st crop.	2nd crop.	1st crop.	2nd crop.
2	Permanent pasture throughout.									
1	Colza.	Maize, aut.	Wheat.	Bulato.	Maize.	...	Wheat.	Bulato.	Fallow.	
1	Flax.	Maize, aut.	Wheat.	Bulato.	Fallow.	...	Flax.	Maize, aut.	Rye.	Bulato.
1	Grass.	...	Flax.	Maize, aut.	Rye.	Bulato.	Maize.	...	Wheat.	Bulato.
1	Wheat.	Bulato.	Fallow.	...	Flax.	Maize, aut.	Rye.	Bulato.	Maize.	
1	Rye.	Bulato.	Maize.	...	Colza.	Maize, aut.	Wheat.	Bulato.	Maize.	
1	Wheat.	Bulato.	Maize.	...	Wheat.	Bulato.	Maize.	...	Colza.	Maize, aut.
1	Maize.	...	Colza.	Maize, aut.	Wheat.	Bulato.	Fallow.	...	Flax.	Maize, aut.
1	Maize.	...	Rye.	Bulato.	Maize.	...	Colza.	Maize, aut.	Wheat.	Bulato.
10										

N.B.—The Maize, when not mentioned as autumnal, is spring Maize.

It is drawn up to suit a holder of five acres, with his family a cattle. The quantity of maize produced is one-third more th that from land irrigated on the old system, and is sufficient support the family. The amount under pasture is as large can be conveniently arranged, in order to secure as much man for the soil as possible, and, in the case of a five-acre plot, support two cows. The wheat and rye grown will pay the ren the ground to the proprietor, and the spring hay the whole of irrigation, leaving the remaining crops to the holder entirely fi The same rotation is suitable to a holding of any size, worked one family, the basis being the proportions of grass, grain, other crops, which are, taking the whole in ten parts:—t tenths permanent pasture; one-tenth grass crop; three-ter wheat and rye; two-tenths to spring maize; two-tenths autumnal maizes and oil-yielding crops.

It will be noticed that neither rice cultivation nor marcite cu vation—the well-known flooded winter grass crop of Italy—en at all into the above proposition, being generally excluded f the proposed irrigational demand. This is highly significant, appears to point to the conclusion that such cultivation is ra on the wane in Italy. Probably it is not economical on well-far lands; the winter grass crop is believed to yield only a qua more through flooded irrigation on the marcitorial system, both this and rice cultivation are considered injurious to public health in Lombardy, having been for many years forbi within certain distances of towns, cities, and villages. In Port lands formerly growing rice are now otherwise cultivated, economic grounds; experience plainly showing that the produc of other grain, and the support of cattle, are more remunera In this special instance, as returns are obtained from using water for motive purposes, driving mills, &c., it is also extre probable that it is not only more convenient, but also more re nerative, to use water during the winter months in that way.

With regard to the injuriousness of the neighbourhood of ltivation, or of any swampy cultivation, there is still consider oubt. In India rice has been grown close to numerous mili cantonments for many years, without any detrimental effe *whereas, the neigh*bourhood of a single rice patch in a fork in

is sometimes almost deadly, and snipe shooting for a few
over rice fields in China and Ceylon is almost certain to
se fever. Medical men have given widely opposed opinions
this subject, as well as on the effects of irrigation gene-
ly; from which, apparently, the only sound conclusion seems
be, that irrigation, properly conducted, is perfectly innocuous,
d that it is only when the drainage of the country is allowed
stagnate for a long time that injury results. This will per-
ly explain how it is that rice cultivation may or may not
injurious, as in some cases the water is allowed to stagnate,
changed and without flow, for a very long time—a perfectly
necessary proceeding, which, producing an organic decay more
pid under high temperature, is the cause of noxious miasma.
would hardly seem, however, that in this special case hygienic
sons alone would stop marcite and rice cultivation—as it need
be carried on near villages—but rather reasons of economy.
h a conclusion would, therefore, show that water is more
nomically expended on other crops, and that irrigation-water
hence becoming more valuable than formerly.

With reference to the amount of water necessary per acre of
gated land in the tract under consideration, the following four
les supply the data on which it has been based:

TABLE I.

olume of water in cubic feet necessary per acre at each watering.

	Absorbed.		Utilized.		Expended.	
	Meadow.	Arable.	Meadow.	Arable	Meadow.	Arable.
Regis	5 885	8 476	8 665	9 323	14 550	17 799
ntoni	5 585	8 476	8 411	8 454	14 296	16 930
ommittee of Engineers	5 885	8 476	10 404	11 314	16 288	19 790
Totals	...	...	27 480	29 091	45 134	54 519
Mean	5 885	8 476	9 160	9 697	15 045	18 173

TABLE II.

Quantity of continuous water in cubic feet per second per acre necessary for irrigation.

	Meadow land.		Arable land.	
	For watering once in 7 days.	For watering once in 10 days.	For watering once in 14 days.	For watering once in 20 days.
De Regis	·02404	·01683	·01471	·01029
Cantoni	·02362	·01653	·01398	·00978
Committee of Engineers ...	·02691	·01883	·01635	·01008
Total	·07457	·05219	·04504	·03015
Mean	·02486	·01740	·01501	·01005

TABLE III.

Area in acres that can be irrigated by one cubic foot per second.

	Meadow land.		Arable land.	
	Watered once in 7 days.	Watered once in 10 days.	Watered once in 14 days.	Watered once in 20 days.
De Regis	41·46	59·22	67·78	96·83
Cantoni	42·19	60·28	71·26	101·80
Committee of Engineers ...	37·07	52 91	60·95	98·86
Total	120·72	172·41	199·99	297·49
Mean	40·23	57·47	66·66	99·16

TABLE IV.

Supply necessary for each acre of the irrigable area.

	Area in acres.	Sandy soil.		Clayey soil.	
		Quantity of continuous water necessary for one acre in cubic feet per second.	Product.	Quantity of continuous water necessary for one acre in cubic feet per second.	Product.
		cub. ft.	cub. ft.	cub. ft.	cub. ft.
Meadow	1·48	·02486	·03679	·01740	·02575
Arable (?) ...	1·98	·01501	·02972	·01005	·01990
Corn (?)	1·48				
Quantity for ...	4·94	=	·06651	or	·04565
Quantity for ...	1·00	=	·01346	or	·00924

Result adopted for calculation of supply to one acre: in sandy il, ·01346 cubic feet: in clayey soil, ·00924 cubic feet.

To complete the calculations of this part of the subject, before tering into details and comparisons, it may be said that, dividing e total area under command into two classes, sandy and clayey ils, the total water supply required is as follows:—

	Cub. ft. per sec.	
For 47 674 acres clayey at ·00924 cubic feet =	440	
,, 143 016 ,, sandy at ·01346 cubic feet =	1925	
Total 190 690 Total	2365	
Deducting an already existing supply of ...	310	
Adding for irrigation in a lower tract of land	388	
Supply required ... ... =	2443	c.m.
Hence the actual supply of the canal is fixed =	2825	or 80.

As the whole tract amounts to 216 234 acres, this would show more than seven-eighths will be irrigated, and, taking the quanes approximately, the average supply over the irrigated area is 12 cubic feet per second per acre, or is such that 1 cubic foot per ond will irrigate 90 acres, from which, according to Table III., average watering will be once in 18 days or 20 in a year.

Before entering into these general quantities, the principles and ails on which they are based require examination.

In table I. the quantity of water sufficient for one irrigation or tering is taken at 15 000 cubic feet for pasture, and 18 170 for ble land; it cannot be doubted, by any one conversant with irriion in India and Spain, that this quantity is excessive; the lians of both Piedmont and Lombardy have for a long time been ceedingly wasteful of irrigation water; they have had the unusual vantages of being able to get as much as they like, and as adtted by themselves in Piedmont, the waste is excessive, a natural ult of having been provided with too much; in Lombardy in, those that dare to raise their word in private against the ditions of the past have expressed strong opinions that water can re be made to perform a much higher duty than at present.

The object of ordinary irrigation in hot climates is simply to pply the place of rain and soften the soil, and differs much from the irrigation of lands in colder regions, which, partaking the nature of sewage irrigation, has for its object the deposit of a fertilizing sediment rather than a supply of moisture, and c responds in Italy to maricitorial and rice cultivation only. T latter description of irrigation being excluded from the project a data under consideration, the former alone has to be dealt with, a for such purposes in India and in Spain a watering of 10 000 cu feet is ample, and would doubtless be enough in Italy also, either f pasture or arable land. One such watering represents a depth of feet over an acre and is equivalent to a continuous supply through the year of ·000317 cubic foot per second. It may be said th under different states of climate, soil and subsoil, more water wo be required even in hot climates, but to this the reply is that a grea number of waterings might be required, but not a larger supply each watering. In support of the statement that 10 000 cubic f would be sufficient, it may be noticed that the learned and scienti Professor Cantoni, Director of the School of Agriculture at Mil who has been continually and is still prosecuting researches int agronomic and agricultural matters, fixes his quantities low than the previous data of the older Italian hydraulic engineers, an far lower than those of the Commission of Engineers (about on eighth less); it is possible also that, were he not an Italian an holding a Government appointment, he might be very much bold in his reduction.

With regard to the number of waterings, the amount allow appears, according to Tables II. and IV., to be 52 and 26 in th year for meadow and arable land respectively on sandy soils, and and 18 on clayey soils; but, as the canals are closed for cleansin and repairs during April and October, these numbers are reduc in practical application to 46 and 23 for sandy, and 30 and 15 clayey lands. Now, leaving out of consideration the fact that the waterings are a half and three-quarters larger than would be requ site in India or Spain, their number seems excessive. In India th number of waterings prescribed on the Nageenah Canal, Nort West Provinces, is thus (*vide* "Hydraulic Manual," Part II.): F fruit gardens, 8 per annum; for hemp, 5 per crop; for rice, indig sugar, tobacco, grasses, and herbs, 4 per crop; for cotton, whea barley, and grains and pulses, 3 per crop. In Spain the numb

aterings in the year generally necessary are, according to Mr.
erts's excellent pamphlet : For corn, flax, potatoes, olives, and
, 6 waterings ; for meadows and artificial grass, 8 ; and for garden
luce, 20 ; and these by no means show the highest duty obtained
ater in Spain, for, in the clayey vegas of Alcanadre and Lodosa,
lens are irrigated with ·0014 cubic foot per second per acre
ugh the year, and only require double or treble that amount,
·004 cubic foot per second in very dry seasons ; whereas the
ring of garden land with twenty irrigations mentioned above
ires ·012 cubic foot per second per acre. In both Northern
Southern India ·01 cubic foot per second per acre is considered
ll and liberal gross allowance for all crops, except rice and crops
n on the flooding or marcitorial principle, where sediment is
object, while the net allowance per acre yearly appears to be
t from one-half to three-quarters of that amount.

he inevitable conclusion, therefore, appears to be that Italian
tice gives one-half too much water at each watering, and at
one-half too many waterings, thus employing in detail more
double the water that is necessary according to both Indian
Spanish practice, the conditions of soil and climate being more
urable in Piedmont and Lombardy than probably in either
in or India.

ith reference to the water supply in the gross, or water duty
the whole tract of land, the ultimate duty reached in clayey
, according to this project, is 110 acres to a cubic foot per
ond. On previous old works the duty reached in Piedmont and
bardy seems to vary from 60 to 110, 90 and 100 being the
e favourable cases, and 60 to 80 the more usual. In India the
r duty arrived at was on the Eastern Jumna Canal, in 1864,
acres ; on the Western Jumna Canal, in 1863, 280 acres ; and
the Ganges Canal, in 1864, 140 acres ; and these on canals that
not fully developed, thus pointing to a safe gross water duty,
luding single waterings, of double that obtained in Italy. It is
ortunately useless to mention these things to Italians, whose
s of hydraulic grandeur and authority are confined to the Nav-
Grande and their old hydraulic authors and engineers ; to say
them that there is a canal from the Ganges that is designed to
y a volume of 7000 cubic *feet, or 198* cubic metres per second, is

even now unwise, while to attempt to explain that irrigation is n
only Oriental in origin, but that ignorant natives of India, led
military men who cannot be called engineers in the civilized Weste
sense of adepts at scientific construction, but whose proper sphere
the siege and the battle-field, have, in spite of a wonderful chain
mistakes, succeeded in carrying out, not only the largest works
irrigation, but also the most economic distribution, would be i
tensely absurd.

The increase of produce due to irrigation in the tract under co
sideration has been calculated by a commission nominated by t
College of Engineers of Milan, acting on behalf of the Governm
of the country granting the concession. Knowing the way in wh
petty intrigue enters into every matter in Italy, one cannot in t
case, any more than in determining the amount of water necess
for the crops, expect unbiassed data. Under similar circumstanc
in England, no one would think of curtailing the profits
hampering the undertakings of engineers in this manner; on
contrary, one would think, the greater the profit and freedom,
more likely would be the extension of similar works conducive to
public good as well as to private interest in every way; petty id
however, seem to rule in Italy. The data, however, are interesti
and may possibly, after all, be accepted as tolerably correct.
same amount of area has the value of its produce compared un
dry and under wet cultivation, and the difference credited as
result of irrigation. The land is divided into four classes acc
ing to the degree of sandiness, and the results are given. Th
for the extremes of sandiness and clayeyness are alone give
detail; they are as follow:—

Dry Land.	acres.	per acre.	product.	s.	d.	£	s.	d.	£	s.	d.	
a. Wheat	1·235	9·023	11·143	4	10	2	12	6				
b. Rye	1·235	12·545	15·403	3	7	2	14	8				
c. Maize	2·471	18·157	44·866	3	0	6	12	0				
d. Maize, quarantine, 2nd crop	1·235	9·023	11·143	3	0	1	12	9				
e. Straw, 2nd crop	1·235	C 5·985	C 7·391	0	9	0	7	7				
f. Mulberry leaves	4·942	C 4·803	C 23·736	1	6	2	8	10	16	8	4	
Deduct for disasters ⅑th	...	...	...	...		1	16	6				
„ drought ⅐th	...	..	...	...		2	3	0	3	19	6	
Produce of	4·942	...	...	...		...			12	8	10	
Irrigated Land.												
1. Wheat	1·483	9·023	13·381	4	10	3	3	0				
2. Flax	·494	...	...	81	10	2	3	2				
3. Colza	·494	11·775	5·817	7	6	2	2	8				
4. Maize	·988	33·562	83·159	3	0	4	17	8				
5. Maize, autumnal, 2nd crop	·988	25·309	25·005	3	0	3	13	8				
6. Pasture, three cuttings	·988	C 42·052	C 41·547	1	8	4	14	0				
7. Grass	·494	C 42·052	C 20·773	1	8	2	7	0				
8. Straw	1·483	C 11·969	C 17·750	0	7	0	14	8				
9. Erba quartirola	2·965	...	...	9	1	1	8	9				
10. Mulberry leaves	4·942	C 5·591	C 27·631	1	6	2	16	9	28	1	4	
Deduct for disasters generally ⅑th ...	...	...	...	...		1	17	8				
10. „ „ ⅐th ...	...	...	...	...		0	8	1				
6, 7, 9. „ „ 1/15th ...	...	...	...	...		0	11	3	2	16	7	Net.
Produce of	4·942	...	...	...		...			25	4	9	Net.
Increase due to Irrigation	4·942	...	...	...		...			12	15	10	Net.
„ per acre	1·	...	...	...		...			2	11	9	

TABLE OF PRODUCE.—*Clayey Soil.*

		Price in Bushels or Cwt. (C).		Price.	Value.		
Dry Land.	acres.	per acre.	product.	s. d.	£ s. d.	£ s. d.	
a. Wheat	2·471	15 076	37 253	4 10	8 15 4	...	
c. Maize	2·471	24·209	59·820	3 0	8 16 0	...	
d. Maize, quarantine, 2nd crop	1·235	12·104	14·948	3 0	2 4 0	...	
e. Straw 2nd crop	1·235	C 5.985	C 7·491	0 9	0 7 7	...	
f. Mulberry leaves	4·942	C 3·543	C 17·510	1 6	1 16 0	21 18 11	Gross.
Deduct for disasters $\frac{1}{9}$th	...	...	...	...	2 8 9	...	
" drought $\frac{1}{6}$th	...	...	...	...	1 18 0	4 6 9	
Produce of	4·942	...	...	...	...	17 12 2	Net.
Irrigated Land.							
1. Wheat	1·483	15·076	22·358	4 10	5 5 2		
2. Flax	·494	...	...	90 10	2 8 0		
3. Colza	·494	13·425	6·632	7 6	2 8 9		
4. Maize	·988	36·313	35·877	3 0	5 5 7		
5. Maize, autumnal, 2nd crop	·988	26·850	26·528	3 0	3 18 1		
6. Pasture, 3 cuttings	·988	C 42·052	C 41·547	1 8	4 13 11		
7. Grass	·494	C 42·052	C 20·773	1 8	2 6 10		
8. Straw	1·483	C 11·970	C 17·751	0 7	0 14 7		
9. Erba quartirola	2·965	...	...	9 1	1 8 9		
10. Mulberry leaves...	4·942	C 4·173	C 20·623	1 6	2 2 4	30 12 5	Gross.
Deduct for disasters generally $\frac{1}{9}$th ...	...	...	...	...	2 4 6		
10. " " $\frac{1}{7}$th ...	...	...	...	...	0 6 0		
6, 7, 9 " " $\frac{1}{15}$th ...	...	...	...	...	0 11 4	3 1 10	
Produce of	4·942	...	...	...	...	27 10 7	Net.
Increase due to irrigation	4·942					9 18 5	Net.

The gross result from these data is the following increase of value crop due to irrigation for the four classes of land, namely:

	£	s.	d.	Mean.
1st for the most sandy—per acre	2	11	9	£2 4*s.* 8*d.*
2nd ,, ,, do.	2	5	0	
3rd ,, ,, do.	2	2	0	
4th for the most clayey do.	2	0	0	

The results last given seem very small; but it must be noticed t very low values are given to the produce, in all cases only ree-fourths the mean local value for the preceding five years ing applied; but they are useful in showing how little results y within the area under consideration. The principal point of portance seems to have been, purposely or otherwise, entirely itted; the yield per acre of wheat is assumed to be the same both der irrigation and otherwise. The maize, the staple food of the asantry, is assumed to yield more than a half more, and the mul- rry crop of leaves one-seventh more; but the wheat is supposed t to be affected by irrigation. Now in India, where crops are grown pendent on the annual rain, as well as assisted by irrigation, the crease of yield of grain due to irrigation is large; cereals and oil eds yielding from a quarter to a half more.

There can therefore be little doubt that there must be some crease of yield of wheat also in Italy, and it appears unfortunate at any profits due to irrigation in any way should be allowed to ss unnoticed. If the object is to show as little profit to the ndholder, and hence obtain water at as a low rate as possible by milar devices, it is a very shortsighted one, on behalf of the overnment of a country, the sole result of depreciating the value d profits of irrigation being to leave the country unirrigated and an impoverished agricultural state. Taking, however, the figures ven by the Commission as relatively correct, these show that the fect of irrigation is to more than double the value of the produce sandy soil, and to increase it by nearly two-thirds on clayey soil, en the improved rotation of crop is adopted; and at the same ne prove clearly that, if allowance be made for an increase in the oduce of wheat per acre, and for higher rates for values of produce nerally, the value of produce due to irrigation is fully doubled on the lands that benefit least by it. The importance of so i vertible a fact requires no comment, and it becomes a co basis for calculating what amount of water-rate could be eas under a more correct valuation of produce. Taking, howe present valuation, which gives as an increase of value £2 5*s.* as a mean, though, perhaps, it would be more corre sume £3, to determine the water-rate per acre adopted, viz., and 7*s.* 5*d.* for sandy and clayey soils, respectively, it woul that the water-rate is fixed at a price about one-fifth of the of value resulting from the aid of the water, the landed pr incurring at their own expense the costs of levelling an their land for irrigation, and keeping their own trenches o bution in a proper state of maintenance. This is doubtles low water-rate; but the circumstances under which this i project is being carried out are peculiar, and the terms of cession are drawn up to suit the case. But of this mo after.

The following is an abstract of the cost of the works of t Maggiore project:

Construction.

Headworks	£25
No. 1. Main canal 30 miles, section 604 square feet	215
2. „ 14¾ „ 320 „	12
3. „ 18¼ „ 341 „	102
Secondary canals, 132 miles in all	83
Keepers' houses	2
Legal expenses	15
Engineering expenses...	17
Interest	50
Miscellaneous	5
Total ...	£880

[redacted]

Maintenance Annually.

Headworks	£1 271
Main canal 63 miles	1 822
Establishment and office	2 400
Imposts	4 507
Total ...	£10 000

Expense per Acre to Landed Proprietors.

	£	s.	d.
Land occupied by trenches	1	7	0
Excavation in trenches		8	0
Buildings	1	0	0
Adapting the land	1	2	0
	3	17	0
Annual maintenance of trenches and administration per acre		1	3

Details.—The headworks, which include a large basin, a large navigation lock for communication with the Ticino, a roadway and sluices, do not seem to have any features worthy of remark. The main canals are constructed to deliver, No. 1, 2825 cubic feet with a fall of ·20 and ·15 per 1000; No. 2, 1766 cubic feet with a fall of ·1 per 1000; besides 209 feet by 26 falls or locks; No. 3, 530 cubic feet with a fall of ·13 per 1000; some portions are paved in boulder work set in earth, or in ordinary lime, and in some cases in hydraulic mortar over a bed of béton, with walled sides. The works on the three main canals consist of:

2 railway and tramway bridges.
7 bridges for provincial roads.
55 smaller road bridges.
13 over crossings for rivers and brooks.
27 locks, mostly with bridges or outlets.
56 falls, syphons, and under passages.
9 keepers' lodges.

The secondary canals are 16 in number, of different lengths and sections; they are generally of four sections.

		Feet.		Cubic feet per second.
No. 1.	With a bottom width of	13·1	carrying	103
2.	„	9·8	„	78
3.	„	6·6	„	54
4.	„	3·3	„	30

They have an inclination of ·5 per thousand; and the works consist of 38 bridges and falls for provincial roads, 395 district road bridges, and 397 petty bridges.

The capitalised price of the water in the Lago Maggiore scheme is fixed thus for total amounts:

	£	*s.*	*d.*
Continuous water, per cubic foot per second ...	589	4	7
Summer water	566	11	5
Winter water	22	13	2

Separating this into payments over the forty years in which the project is to repay its costs, and allowing for 6 per cent. it becomes:

	£	*s.*	*d.*
Continuous water, per cubic ft. per second (yearly)	41	7	2
Summer	39	13	2
Winter	1	14	0

And under the agricultural rotation adopted, with the quantity of water necessary for each acre of sandy and clayey land, the price of water per acre is:

	£	*s.*	*d.*		*s.*	*d.*
Sandy, capitalised	7	14	5	yearly	10	8
Clayey, „ „	5	5	11	„ „ „	7	6

Checking the capitalised result per acre as follows:

	£	*s.*	*d.*		
Sandy 143 016 acres at	7	14	5	yields	£1 104 083
Clayey 47 674 „ „	5	5	11	„	252 672
				Total ...	£1 356 755

that the capitalised value of the irrigation effected per han covers the costs of construction and maintenance s, which is £1 280 000: a further check on this is capitalising the value of the water per cubic foot. The al supply of the canal will be as before stated, 2825 but as during the first two years the amount to be ccording to the concession, only 1558·9 cubic feet of d 1059·5 cubic feet of winter water, from which 5 per o be deducted for loss by infiltration, the capitalised ld be:

£ s. d.
water 1476 cubic ft. at 566 11 5 = £836 257
,, 1006 ,, ,, at 22 13 2 = 22 793
Total ... £859 050

nual return under the same circumstances:

£ s. d.
water 1476 cubic ft. at 39 13 2 = £58 535
,, 1006 ,, ,, ,, 1 14 0 = 1 710
Total ... £60 245

s for navigation are calculated on the basis that the compete successfully with the railway, when carrying alf the present railway rates; and, applying this to a f 40 tons sent by either manner, the navigation toll is shillings per boat load, or about 3½*d.* per ton: it is calt such a boat would make 35 voyages in a year going the current, but requiring one or two horses to tow it up y or full. On these principles the expected return from is estimated at £12,000; by others as follows:

Tatti, engineer	£15 400
Conte Annoni	15 200
mmission of the College of Engineers ilan	1 800

parison of these data seems, by the evident underrating s of navigation in the last one of them, to throw light on the over-estimated supply required for irrigation by the Commission of the College of Engineers of Milan, and strengthens the belief before expressed.

The returns for motive power are not estimated, as it is probable that some time may elapse before it is utilized at all; but the amount of motive power is thus calculated: 26 falls on 28 locks having a total fall of 210 feet, having a supply of water, diminishing from 494 to 211 cubic feet, or a mean supply of 353 cubic feet per second, will give 200 horse power a[illegible]ch fall, or 5000 horse power in all on the main canal, and in the [illegible]ame way 1000 or 2000 horse power more on the secondary canals. It will be serviceable for threshing corn, spinning silk, co[illegible] and flax, paper, cloth, and other manufactures. Other returns [illegible]ay also be obtained from turf and grass-cutting, water for dome[illegible] use, wash-houses, and the supply of drinking cisterns fo[illegible]. The total annual return that may be expected amounts to:

By irrigation	£60 245
By navigation	12 000
By other sources	3 755
Total ...	£76 000

Considering, then, the total cost of the works to be	£880 000
And the annual cost of maintenance to be ...	10 000
And deducting this from the annual return of ...	76 000
The remainder	£66 000

represents an interest of 7½ per cent. on the total cost.

It may be interesting, before entering into comment on the subject of cost and return, to deduce the profit per acre that the occupiers of the land can obtain on the whole, assuming the previous data of increase of yield and needful supply of water as the basis of calculation.

The expenses per acre to the landed proprietors capitalised in the *foregoing* data may be reduced to annual payments over the y years, allowing for an interest on the capital of 6 per cent., become thus:

	s.	d.
Land occupied by trenches	1	9
Excavation	0	6
Buildings	1	4
Adapting the land	1	5
Maintenance of the trenches } annual	0	5
Administration of all sorts } annual	0	10
Total ...	6	3

and the profit per acre is then:

	Cost of water.		Expenses on the land.		Total.		Value of increase of produce.			Profit per acre.		
	s.	d.	s.	d.	s.	d.	£	s.	d.	£	s.	d.
For sandy soil	10	8	6	3	16	11	2	8	6	1	11	7
For clayey soil	7	5	6	3	13	8	2	0	7	1	6	11
Mean	9	0	6	3	15	3	2	4	6	1	9	3

Besides this profit, the landholder is much benefited by the effect of irrigation, as the labour of ploughing, harrowing and hoeing is much reduced, and again, as so much land is under pasture, his labour there is reduced to nothing; the soil also becomes much improved in time, and the yield again increased beyond the amount calculated: for these advantages the landowner can again demand justly from him an increased rent; and the capitalised value of this increase of rent will be eventually shown in increased saleable value of the land. It is extremely unfortunate that no data are forthcoming on either of these points, especially as there is such a vast extent of land in Northern Italy that has been brought under irrigation at different times which could have well supplied, at least approximately, sufficient information to have given a sound basis on which to rest expected results of this nature. It seems indeed extraordinary that Signor Villoresi, the engineer of the Lago Maggiore project, who has evidently spared no pains in procuring and setting forth so much detail bearing on his scheme, should have neglected to enter into such an important source of return.

To every irrigation undertaking there are three direct and legitimate sources of return.

1. The profit to the shareholders, justly due to them, the capitalists, directors, and engineers, obtained by charging more for the water than it actually costs them, although far less than its value as shown by results.

2. The profit to the landholders or occupiers, whose increase of yield, and hence increase of profit, after paying the water-rate fixed, is due to the supply of water to the land in the first instance.

3. The profit to the landowners by the improvement of their property and land, from the continuous effect of irrigation, and the advantages of having water available.

Besides these, the indirect advantages are innumerable, having their effect on the people and nation generally, as well as on other nations; but these do not admit of calculation: the three direct sources of return, however, do; and it is solely by means of a careful investigation of their results that the true value of the water can be arrived at, with reference to and in proportion to which, and not according to the haggling with the users of the water, a just water-rate can be determined; the success of the irrigation project being principally shown again by a comparison of the cost with the true value of the water.

Failing, therefore, to obtain information on the increase of value of land due to irrigation in Northern Italy, the following data for Spain, from Mr. Roberts's pamphlet, will give some indication of what the increase of value might be:

	Dry per acre.	Irrigated per acre.
Rioja district	rent 9*s*. to 12*s*.	£9·5 to £10·2.
Zamora, Castile	value £14 to £18.	£35 to £41.
Near Madrid, 1st class land value	£32	£128
,, ,, 2nd ,, ,,	£20	£100
,, ,, 3rd ,, ,,	£12	£72
,, ,, 4th ,, ,,	£6	£60
Ampurdan, Cataluna ,,	£100	£200 to £300
Spain generally, 1st class land inc. of value		100 to 200 p. c.
,, ,, inferior land ,,		1000 to 1500 p. c.

ald indicate that it is most probable that the value of the
rthern Italy would be at least doubled by irrigation.
, again, where canals have, at least in a very incomplete
state, existed for many years, the profits to the land-
very plainly shown. A large portion, if not all, of the
tracts watered by the Ganges Canal and the Eastern
ern Jumna Canals are, like most of the land in Oriental
, the actual property of the Crown or government of the
and the rent of the land in these tracts is newly fixed after
riods—five, seven, or eleven years—the enhancement of
he land as it becomes brought under irrigation being deter-
those intervals and credited to the effects of irrigation, as
he water-rate paid by the occupiers of the soil. Turning
es to "Hydraulic Manual," Part. II., we find, among the
iven by canals :

		By water rate. £	By enhancement of rent. £	Total returns. £
Jumna Canal in	1846	12 175	14 965	27 140
Jumna Canal	1845	29 888	37 000	66 888
Canal	1867	136 353	80 018	216 371
Canal	1868	244 156	161 260	405 416

show that in two out of the three great canals the
ment of rent is a larger source of return than the water-
that it is only on the least developed canal of the three
is less, and even then amounts to two-thirds of it. This
ubt, under what we should call in European countries an
al state of affairs; and it is evident that under such
ances, where the owners of the works and the water are
owners of the land, they could, if they preferred it, give
r gratis, and raise the whole of the returns by means of
ment of land rent. In Southern India, unfortunately, the
prevails, of throwing into one payment the water-rate and
rent, so that one is unable to distinguish between the two
f returns.
he whole, however, these figures, as well as those for
how incontestably that the landowner makes an immense

profit from the results of irrigation; whereas the water owner has to haggle over a petty water-rate with the occupier, in order to make it possible to carry out the works at all; or, in other words, every one profits highly from the water except those through whose skill and management the water is supplied; and, more, it seems likely that this state of things will continue until the immense profits of irrigation are fully set forth in such a way that ignorance of them can no longer be professed. When this is done, a more adequate water-rate can be demanded, and will be cheerfully paid by the occupier, and a second water-rate should also be demanded from the landowner.

The necessity as well as the justice of a second water-rate from the landlord has been very recently shown in India, although, of course, much opposition was made. There are certain districts in India where the soil has been either alienated from the Crown by gift at some period, or has been put under a permanent settlement of land rent in perpetuity, that cannot be enhanced. In undertaking works of irrigation in such districts, the Government saw itself deprived of the main source of return obtained in other cases, through the right of landownership, and the difficulty was, therefore, met by an Act of the Imperial Government at Calcutta in 1870, drawn up by General Strachey, Inspector-General of Irrigation, which, among other matters connected with the subject, ordained that a water-rate should be paid both by landlord and occupier, and besides, that a certain small water-rate should be paid by those owning or holding land within an irrigable area, but declining to use the water or sell their land. This Act marks an era in irrigational matters, and points the way to the rest of the world by which carrying out irrigation projects may be rendered, as they should be, sufficiently remunerative to those that undertake them; much praise, therefore, is due to the then Inspector of Irrigation for carrying out such a measure, which must have originally met with great opposition in a country like India, where the natives will haggle over giving a halfpenny or a penny for every pound one may put into their pockets, and where the English, having generally falsely so-called liberal notions, would, in most instances, not understand the justice and true liberality of such measures.

These principles and facts may be said to have established for future, in Europe and elsewhere, that a second water-rate can justly demanded from the landlord after the land is fairly ught under irrigation, or, if alternatively, that the occupier's ter-rate can be increased, so as to include the two rates in one yment, leaving him to settle his own proportion of it with the downer.

Having thus pointed out how important an element of profit been neglected in the calculations of the Lago Maggiore ject, having indicated its value, and shown how it might have n raised, let us return to the consideration of the cost and arn data.

First, as to the works themselves and their design, there seems be very little deserving of special comment: from the remarks de in the drawing up of the project, the use of hydraulic ment is treated as a novelty in Italy, and is mentioned as ving been adopted in France and Belgium; this may be considered as the key-note of the indigenous engineering. For asurement of water, the old *modulo magistrale* of Milan, with *ncia magistrale* as the unit of water measurement, remains its pristine state; sluices and outlets are also very primitive. agine ourselves in England at the present time using the mproved locomotive of Trevithick and the cast-iron rails of period, and we can understand the progress of the Italians in ranch of engineering equally important to them as improved mmunication is to us; while, therefore, criticism, on Italian ineering construction is quite out of place at present: this ection, however, does not extend to examination of the results their works of irrigation.

The complete scheme, costing £880 000, will irrigate 190 690 es with 2472 cubic feet per second, out of the 2825 of full supply; wing proportionately for the remaining 353 cubic feet per ond, we obtain, as the grand result of the whole scheme when working order, an irrigation of 217 930 acres, or a tract of 389 are miles, allowing one-eighth as unirrigable, for the total cost hat is, the cost per irrigable acre is a little more than £4, and cost per square mile of irrigable tract is £2256. Referring to erts for similar information for Spain, we get:

Cost per acre.

		£	s.
Province of	Madrid	15	6
„	Logroño	7	15
„	Toledo	6	5
„	Gerona	8	6
„	Leon	7	10
„	Navarra	7	0
„	Guadalajara	6	10

These are the results of carefully compiled estimates, that al for all contingencies as well as for liberal contractors' profits, which there is no mention in the data of the Italian project.

For India we obtain the following results on partly develop canals.

	Total outlay.	Irrigated area.	Tract irrig
	£	Acres.	Sq. Miles
Ganges Canal, 1864	2 058 714	449 788	
„ „ 1870	2 402 438	...	16,000
Eastern Jumna Canal, 1846 ...	81 460	421 875	497
Western Jumna Canal, 1846 ...	119 405	351 501	1 345
Rohilcund Canals, 1864 ...	31 190	83 904	—

These show results of 3, 4, and 5 acres irrigated for £1; an allowing for difference in cost of labour to the very utmo amount, one acre would be irrigated in India for £1, against £ in Italy, or £5 or £6 in Spain.

The differences in cost may perhaps be accounted for in t case of Spain, by supposing that the estimates for the works the are for really good construction in the English style. In t case of India, it may be remarked that the acreage there giv is the sum of the acres irrigated, continuously, in the autum and in the spring; *e.g.*, the total yearly irrigation or acrea of the Ganges Canal for the year 1868, given as 1 078 4 acres, is composed of 60 664 acres continuous, 293 604 sprin and 734 132 autumn irrigation; this must then be carefu borne in mind with reference to Indian irrigated areas; but ev after making allowance for this, the Indian construction seems *far more* economical in prime cost.

With regard to maintenance, the annual cost in the Lago ggiore project is £10 000 for 217 930 acres; for Spain there no available data, but in India we have:—

	Estab. and Repairs.	Acres.
Eastern Jumna Canal, 1846	£7 340	421 875
Western Jumna Canal, 1846	12 584	351 401
Ganges Canal, 1868 ...	75 731	1 078 400

the comparison indicates that, after making sufficient allow-e for difference of cost of establishment and labour, the former ing far less and the latter far more in Italy, that maintenance s more in Italy.

The expense per acre to the proprietors in preparing the land, nches, &c., is £3 17*s.* per acre in Italy, against £6 for corn d, and £12 for garden land in Spain; but this is a matter that ends so much on local circumstances, that the comparison is of e value. Nor again is the point of expense to the occupier great importance. In most land fit for irrigation the expense not be very heavy; the work is done by the occupier and his ily or field hands during the time that would otherwise be ccupied, or at least comparatively so; and the labour expended more than counterbalanced in perpetuity by the saving of work the operations of ploughing, harrowing, and hoeing on irrigated and, which is considerably less than in dry land.

The data of cost of all sorts, taken with reference to the age, do not thus indicate any advantages in point of economy avour of Northern Italy over Spain and India; it has not, of se, been possible to obtain strictly corresponding data, but it been shown quite possible to draw undeniably just compari- from those given, after making due allowances.

may be urged that, to relieve the comparisons entirely from doubt regarding acreage, it would have been better to keep them ely in terms of cost, price, value, and return per cubic foot econd of supply; of this there is little doubt, and it would been so arranged had sufficient data been available in that ; there are, unfortunately, none forthcoming for Spain, and r Indian returns, in many cases, the terms, cost, price, and are used without *proper discrimination*: taking these returns rigidly, the cost should be the expense of the works, or capital account, per cubic foot supplied ; the value should represent the whole of the benefits valued and summed per cubic foot, and the price the simple water-rates paid. Such data, however, as can be procured are as follows :—

	Supply.	Total cost.	Cost per cubic foot per second.	Price yearly per cubic feet per second.
	cub. ft.	£	£	£
Lago Maggiore project ...	2825	880 000	307	42
Ganges Canal, 1870	4300	2 402 438	558	44
Eastern Jumna Canal, 1870	956	194 575	206	62
Western Jumna Canal, 1846	2800	119 405	42	24

These are not very instructive, as the supply mentioned is probably not in all cases the supply actually utilised in irrigation alone, and the price yearly may in one case not include, as it should, the amounts obtained by increase of land assessment It must be remembered, also, that the data for Lago Maggiore are those of a completely developed project, whereas in the Indian data they are, excepting the last, those of only partly or imperfectly developed works.

That the water-rate of the Lago Maggiore project, 10*s.* 8*d.* for sandy, and 7*s.* 5*d.* for clayey soils per acre, is very low indeed may be shown by comparison with the following rates in Spain most of which are fixed merely to pay for repairs and guards, the works belonging to the land without having any interest to pay off

	Water-rate per acre yearly.
Canal del Urgel ...	19*s.* 3*d.*
Tagus Valley	10 per cent. of the produce.
Malaga	19*s.*
Lobrigat	5*s.* 6*d.* to 17*s.*
Aragon	4*s.* to £1 7*s.*
Cataluna	12*s.* to 16*s.*
Navarra	12*s.* for four irrigations yearly.
New Canals	1*s.* 7*d.* to 2*s.* 4*d.* for each watering
Frequent custom ...	10 per cent. of the produce.

If 10 per cent. of the produce determined the water-rate on the Lago Maggiore tract, it would be from £2 16*s.* to £3 1*s.*, instead of *7s. 5d. to 10s.* 8*d.*, and this would probably be a fairer water-rate enough has been put forward to show how the project has been dered of its apparent profits, by requiring too much water per , and, besides, by underrating points on which the water-rate the estimate of the value of the results of the water have based.

his happens to be of no importance whatever in this special , as the association carrying out the works consists of users of water, occupiers and landowners, who make and take the whole he profits in whatever shape they may appear; their object is lear themselves of the prime cost of the works in 40 years, retain the works permanently as their own after that time ; and hey can do so by so fixing the rates as to pay only 7½ per cent. he cost, this arrangement suits their purposes. Beyond this, conclusions one would be liable to arrive at with reference to scheme, that 7*s.* 6*d.* or 10*s.* 8*d.*, are just and fair water-rates Northern Italy, and 7½ per cent. is a fair profit on such works e, are evidently false.

these works had been carried out by shareholders not owning olding the land, a really remunerative water-rate of as much half of the value of the increase of produce resulting from ation, which is evidently much more than £2 4*s.* 8*d.* per acre, d be easily paid by the occupiers in the first instance, still ing large profits both to occupiers and landowners, and from latter again the second water-rate might be demanded ; the s themselves might also be sold at some fixed price either to Government or the landowners for a hundred years, having y paid 30 per cent., as the preceding examination has shown. en it is considered that, even then the landholders would be bled to increase permanently the value of the produce of their s by one-half without any risk or investment, it seems extranary that Italian landholders have not already largely invited use of foreign capital for such undertakings and hypothecated lands with this object.

he preceding inquiry into the value of the results of irrigation , it is hoped, have furnished ample evidence of the immense s to all concerned that works of irrigation can produce, and onstrated clearly that it is solely due to a want of careful inves-ion that they *have been so much* ignored hitherto.

3.—THE CONTROL OF FLOODS.

The prevention of the submergence of land by inundations from overcharged rivers, and the drainage from marshes and submerg land of the water that has been allowed to accumulate over it, a kindred engineering problems that appear at first sight to prese but little difficulty. Their theoretical solution, when merely on small scale, is ready and simple; on a larger one, however, the practical details brought into these problems affect them to such degree, that, although the principles involved cannot be said to b subverted, their carrying out is forced into a comparatively n form.

Land liable to submergence from a river is lower than the extrem flood level, and in open communication with it; the remedies consist, therefore, either in lowering the extreme flood level in the channel by providing other passages for the water, partially diverting it, or dredging out a deeper channel, or by warping up the land liable to submergence, or by cutting off possible communication in flood stages between the river and the land by means of embankments. Submerged land, again, remains in that condition for want of sufficient natural outfall; an outfall has, therefore, to be cut, tunnelled, dredged, or enlarged to a sufficient extent to allow gravity alone to do the work, should that be possible or economically sufficient; in other cases pumps are indispensable.

Imagining, then, the case to be one of an area of a few hundred acres, liable to inundation from a river with a moderate declivity, the application of these principles involves generally but little difficulty as regards engineering, and becomes a local economic question, rather than an engineering practical problem. Putting the case again on a large scale, a vast tract submerged by the floods of a river having a very small declivity—the usual condition when large areas are submerged—the dimensions entering into the works that would be necessary in adhering rigidly to the above principles become so large, that their complete execution is positively im*possible in* most cases. Let us adduce the embankments of the ges, the Mahanuddi, the Po, and the levees of the Mississippi, h are not and never can be complete and sufficiently developed sure, by means of themselves alone, the absolute protection of he lands on their banks from the devastating effects of extreme ds.

o this it might be, though perhaps rather thoughtlessly, replied, very extensive works may be so costly as to be impossible, but the application of the principles need not vary. It is, however, oint of fact also a matter of modification of the application of ciple.

he case of a comparatively small river supplying the flood, very ly, and in most cases totally, limits the consideration of the d to its principal point, the extreme flood level; the catchment of a small river being tolerably uniformly supplied throughout rainfall, its upper portions do not require very special consider- ; the declivity of the small river being tolerably rapid, the con- on of the lower ranges of the river does not affect the matter ny very important degree. Remote local conditions being com- tively disregarded, and it being possible to cope with the flood he required point both successfully and economically, the works lved are necessarily small.

n a large scale, on the contrary, the extreme flood level, the re, causes, and duration of the flood may be greatly affected any of the physical conditions of the entire catchment area he region watered by the river and its tributaries, from the est hill on the watershed down to the currents of the ocean, es beyond the river's mouth; and as these physical and meteor- ical conditions vary greatly throughout large countries, a perfect wledge of them as regards the country under consideration is olutely necessary in order to arrive at sufficient information to ble one to propose measures for the mitigation of the effects the flood. In other words, the natural drainage of the whole on under any state or circumstances, as well as everything practically affects it in any way, must be thoroughly known detail.

t will be unnecessary to dilate on the physical laws and ditions of our sphere, matters best understood from studying larger works on *physical geography* to be found in any goo library: and a knowledge of these will hence be assumed. Th detailed knowledge, however, of the physical conditions, and speciall of the rainfall of the region under consideration, may possibly not b obtainable from any book whatever. It is not sufficient to posses meteorological statistics of observations taken at a few towns i the valley of the river, and at one or two points or villages on th hills; it is needful to know definitely what is the greatest amoun of rain that ever falls in the region, the greatest area in it ove which rain falls at any one time, and which portions of the are they are likely to be at any time; or generally how much water, when, and where, so that it may be practically accounted for. Detailed observations taken for many years at a very large numbe of meteorological stations are therefore requisite, and it is almos painful to reflect in how very few instances are even a moderately small number forthcoming. As a notable exception to this apparent apathy, may be noticed the large number of meteorological stations in the United States of America, and the large sum annually spent by their Government in obtaining such information. Besides the meteorological data, a correct detailed topographical and hydrographical knowledge of the whole of the catchment of the river, based on engineering surveys and velocity observations, is necessary in order to determine the discharge and the flood level of the river at any time, and under any possible meteorological condition. Having all this information, we are enabled at any time to state what will be the results in rise and amount of discharge of the river, corresponding to and resulting from any special rainfall lasting for any usual or unusual time over an area, or detached portions of area within the catchment basin, and the evils to be contended with are then fully known before commencing to deal with them and attempting to mitigate their ill effects by means of engineering works of any sort.

To this it may be replied, that the expense of obtaining all these data, and especially those of a hydrographical and topographical nature, which cannot be done except by skilled hydraulic engineers, must necessarily be very large; and if after all this it should be discovered that under any circumstances no engineering works could remove the evils, or even moderate them to an important extent, the expense would have been uselessly incurred.

Not entirely so. Even should no works be attempted, the information can be made use of in the protection of human life, and in thus mitigating the fearful effects produced by sudden and devastating floods. The extent of land liable to submergence under certain conditions of rainfall in any part of the country being known to a practical certainty, the telegraph can be employed to warn the inhabitants of an impending flood, and allow them to save at least their own lives, and perhaps also that of their cattle and movable valuables. It may be urged that the terrible catastrophes resulting in large loss of life generally commence with the bursting of an embankment, which happens before the flood overtops it; doubtless it is so, but it would be an important part of the topographical knowledge to ascertain to what height of flood these embankments, which, when in sound condition, are in most cases only sufficient protection against very moderate floods, are practically safe. Timely warning could, therefore, be afforded in any case, and the inhabitants would be spared the terrible infliction, in case of flood, of watching the waters rising, and not knowing either how much higher they might rise, or to what height of flood their dams might be safe.

But to proceed to the main object, the protection of the land, as well as its inhabitants, when the matter is one of large extent and importance.

The usual practice hitherto, notably in the case of several districts in Holland, seems to have been, to construct continuous lines of embankment along all the existing edges of the various channels of the river, and discharge the waters within them on the flooded land into the rivers by means of pumps. This caused no doubt a certain amount of mitigation of evil up to certain height of flood level only; beyond that, it is sufficiently evident in theory, and has been fully established in practice, that the means employed cease to be a remedy, and become a decided aggravation of the cause of disaster, effecting an excess of external pressure on the embankments. Besides this, as the channels of the river are under these circumstances allowed to silt themselves up, not only the bed level, but also the flood level corresponding to the same amount of discharge, is allowed to rise also; a second aggravation of the evil. Beyond this again, the immense length of these circuitous embankments causes them to be exceedingly costly. These three reasons will, it is hoped, have sufficiently demonstrated the fallacy of employing the means, that are occasionally appropriate on smaller works, to those of large extent.

Before entering into the subject of works based on better principles, let us first examine the conditions of a flood under circumstances that admit of easy personal observation.

Let us imagine ourselves to be standing on the bank of an Indian river, as wide as the Thames at Hammersmith, in a mansun season of unusually high rainfall, the maximum annual rainfall being 74 inches, the day maximum 7 inches. The mansun, or periodic rainy season, has set in tolerably mildly; the river swells, increases in depth and velocity, and is discoloured at first; this afterwards passes away, and the water then runs steadily, tolerably clear. The rain increases in the plains, and the sky gives prospects of a heavy storm in the direction of the uplands of the river. Let us watch the effect. The rainfall of the plains, in fact the downpour all around us, increases the depth and the velocity of the river, but its colour is unchanged, in fact it seems nearly pure. Suddenly a roaring of waters, like that below an overtopped mill weir, is heard, and up stream we notice a white line of foam approaching; three or four minutes, and a flood sweeps by on the surface of the river, like a wall of water 3 or 4 feet in height; all this water is muddy and dark with detritus. The waters after this again rise still higher for twenty-four hours, but are yet muddy; the low-lying lands near the river are submerged. We learn afterwards that a considerable fall of rain has taken place in the uplands of the river, and that towns and villages in the plains have been inundated.

Such is the flood, its subsidence is a matter of less moment; and such is the type of flood to which those causing serious catastrophes generally belong. In this case we fully satisfy ourselves of the rationale of the flood; the lowland water rises steadily and clear, going perhaps one mile an hour; the upland water comes down with a velocity of nearly six miles an hour and charged with silt,—for where else is this velocity and this silt to come from except from its course in the hills?—and tops the lowland water; *the combination* of waters gradually decreasing in speed spread themselves out over the land in the first locality, where the form of channel and banks admit of it, and perhaps in more than one, extending even for miles beyond the natural bed of the river.

How is such a flood to be controlled? Apart from the Dutch principle, already shown to be fallacious on a large scale, there are only two methods, either or both of which can be adopted. The first, the improvement of the whole of the natural drainage lines of the country to such an extent that the velocity of the waters may under such circumstances be increased throughout the whole course of the river, and a little beyond it, into the sea or next large river, and so that the natural bed, thus improved, may be sufficiently large to carry off any previously known flood, without being exceeded. The second, any means of separating the upland from the lowland waters, holding or retarding either the one or the other, or portions of either one or the other, and providing for their discharge either separately in different courses, or at different times in the same watercourse. Let us first indicate the nature of the works requiring execution, when the former principle alone is adopted: the perfecting of the natural lines of drainage.

The ultimate free delivery of the water into the sea, or any way entirely free of the river, is perhaps the most important point of all the low-lying lands on the lower ranges of the river being there more extensive than elsewhere; to insure a free delivery, the main outlet of the river should be carried out to deep water, protected on both sides by banks or jetties, against the shore currents, and so directed as to avoid as much as possible the retarding influence of sea storms; through the delta, also, a single direct channel of properly determined dimensions should be made and protected by embankments; by these means the mass of water will, in forcing its way in this course to the sea, scour for itself a deeper bed at the outfall and throughout the lower ranges of the river, and carry off floods more rapidly, improving the river continually. A further advantage from confining the river to one channel is that of the reclamation of a large amount of land previously occupied by marshes, as well as by the numerous old channels of the delta.

In the middle ranges of the river the works to be adopted are all such as will promote a more rapid discharge: the enlargement of the bed wherever it is contracted or narrowed; the removal of obstacles, rocks, small islands, silt deposits, shoals, or anything that impedes velocity; the straightening of the course wherever it can be done to good effect; the prevention of the deposit of silt in such places as would be objectionable; the deepening or dredging of the bed in the requisite places: the whole course to be put under a regimen that would remain constant generally, and besides continue to improve itself by scouring in contradistinction to its former habits of silting up and causing its flood levels to rise.

In the uplands, all the works which should be constructed are those that have for their object the control of the detritus washed down, and the prevention of its deposit at unfavourable spots. If the silt could by any means be entirely prevented from being carried down into the middle ranges of the river, or into the plains, it would be a great achievement; but this being hardly possible, palliative measures are perhaps all that can be adopted. Besides this, the hills might be covered with thick plantations, which, catching the rainfall, would delay its departure, prolong the duration of the flood, and thus lessen the amount of flood water passing off at any one time, or mitigate the flood.

The necessary works dependent on the second of the principles previously mentioned, would be so greatly dependent on local circumstances that they can only be indicated generally. The separation and control of the water from the uplands can be attained by making storage reservoirs at certain places at the foot of the hills, and running all the water falling on them into these by means of catchwater drains skirting the bases of the hills; from these reservoirs the water can be allowed to escape under control into the main watercourse; or, if practicable, the upland waters may be discharged through very large catchwater drains, independently of any reservoir, into some other collateral watercourse that may be convenient, employing even, if necessary, a separate outlet for the discharge into the sea of the upland waters.

In the case, however, of the main river or watercourse being employed as the outlet for the upland waters, it becomes necessary to separate the lowland waters from them as long as possible. In order to do this, the arterial drainage lines of the plains on each side of the main river require rectifying and improving; their waters then have to be cut off from it, and carried by two canals

[redacted]

he main river as far as some point where it may :harge them into it through regulating sluices, into some artificial reservoirs or lakes. These l insure the additional advantages of perfecting ge of the country, and of having a good water ion.

of the two principles thus described would insure a ınd an effective control of floods under any practi- nces. That such works would necessarily be expen- doubt whatever, but they would still be less costly ctive than the continuous lines of embankment he fallacious principles before quoted; the works mprove the rivers instead of deteriorating with lapse the gain by reclamation and irrigation would, apart ɔllateral advantages, yield a profitable return.

4.—TOWAGE.

ɪxperiments show that the pull on the towrope of a ithin practical limits, proportional to the square of the that it varies widely according to the form of the ɪming then a general formula,

$$R = b\,T\,V^2$$

the resistance in lbs.,

: the displacement of the barge in tons,

the velocity [illegible] the water in miles per hour,

›oefficient de[illegible]n the form of the barge.

en found th[illegible] small and bluff barges of ab

ployed on [illegible]es, and for limits of speed

miles an [illegible] coeffici[illegible] $\frac{1{\cdot}5}{\sqrt[3]{T}}$ or gener

and tha[illegible]ormed [illegible] medium size

$= \frac{0{\cdot}75}{[illegible]}$ [illegible] gener[illegible] 0·170;

[illegible] arg[illegible] good lines, w employed on the Danube wire-rope system, which have a leng about eight times their beam, and are about 287 tons' displacemer

$$b = \frac{0{\cdot}5 \text{ to } 0{\cdot}6}{\sqrt[3]{T}} \text{ or generally about } 0{\cdot}109.$$

The limit of speed for ships will be about 10 miles an hour, a beyond these limits the resistance R would vary with the fourt power of V: but within the assumed limits, calculations may b made on the above data.

The number of horses required to draw a train of barges ma hence be readily deduced. The best performance of a draugh horse working 8 hours per day, is assumed to be at the speed o $2\frac{1}{2}$ miles per hour, when he will exert an average pull of abou 120 lbs.; substituting this value in the above formula, we obtai for the tonnage that one horse will pull at the speed of 2·5 mile an hour in still water,

$$T = \frac{R^1}{0{\cdot}17\ V^2} = \frac{120}{0{\cdot}17\ (2{\cdot}5)^2} = 113 \text{ tons.}$$

In a current, the resistance or the pull upon the tow-line wi increase as the square of the speed through the water, but th horse in this instance moving over the ground is going at less speed than that of the boat through the water; and th is an important distinction, which must not be overlooked i estimating the effect of a current. The mode in which th necessary correction must be effected will be best illustrated b an example.

Referring to the last example, let us assume that the barge 113 tons' displacement encounters an adverse current of 1 m an hour, and it is required to know the reduced speed at whi the horse will then go, assuming him to be performing the sa average work per hour.

In the last case, the said work in mile-pounds was 120 2·5 = 300 mile-pounds per hour; in the present case the p upon the rope will be proportional to the square of the veloci through the water (V), and the pull the horse is capal of pulling will be inversely proportional to the velocity which he is travelling (v); and the difference between the two velocities will be the speed of the current (v_1): we have therefore

$$V = v + v_1 \text{ where } v_1 = 1 \text{ mile per hour}$$
$$R = {\cdot}17\, T V^2$$
$$\text{and } Rv = 300 \text{ mile-pounds per hour}$$
$$V^2 (V \pm v_1) = 15{\cdot}4$$

whence $R = 19{\cdot}4\, V^2$, and $V^3 - V^2 = 15{\cdot}4$.

Solving which we obtain $V = 2{\cdot}86$ miles per hour, the speed of the boat through the water;—and the speed past land, or rate at which the horse is going, will be $2{\cdot}86 - 1 = 1{\cdot}86$ miles an hour.

It will be observed from this example that the influence of the current is relatively less important when horses are employed, than when steam-tugs, either paddle or screw, are used, the reason being that in the latter case the reaction operates upon the moving current, whilst in the first case against the immovable tow-path. Thus, in the present example, if the horse instead of being an animal moving on the tow-path had been a steam horse in a tug, the speed through the water would be the same, whether the water was still, or ever so rapid a current. In this instance 2·5 miles an hour the speed past the land, which is the useful result, would be reduced to 1·5 miles an hour in the case of the tug, instead of to 1·86 when horses are used.

The difference of conditions will be more strongly marked if we assume the current to be 2·5 miles an hour, because then it is obvious that the steam tug, capable of moving through still water at that rate, would simply maintain its position if it encountered such a current; and although the paddle-wheels or screw would be revolving at the same rate as before, the only result of their effects, namely, the maintenance of position of the boat would be equally attained if she dropped anchor; in short, the whole power exerted would be thrown away. In the instance of the barge towed by horses, on the other hand, the whole power exerted would be utilized; and it may be shown by the same reasoning as in the last example, that the 113 ton barge would be towed by one horse against a current of 2·5 miles an hour, at the rate of $1\frac{1}{8}$ miles an hour.

Obviously the same reasoning would apply, whether the motive power on the tow-path were horses or a locomotive, or whether the tow-path were dispensed with, and a rope were laid down in the bed of the river, and coiled round a drum in a steam-barge in the manner now generally admitted to be the most economical mode of conducting heavy traffic at a slow speed in rivers of rapid current and on still-water canals.

From the above we may conclude that, in order to tabulate for the effect of a current on the diminution or increase of speed of a horse, we have to calculate the increased or diminished value of V the velocity through the water, and apply it in the general formula—

$$R = bTV^2$$

inserting different values for the constant *b*, which lie between ·109 and ·369, according to the form of the barge.

In the above case R = 120 lbs. for a draught horse; but for other animals corresponding values of R, with reference to their best continuous speed, can be applied.

Assuming a case of a current of 3 miles an hour, and that the ordinary limits for the speed of the horse in towing a load with and against stream, are 4 and 1 mile an hour respectively, the velocity through the water becomes 1 and 4 miles an hour, and the loads 706 and 44 tons, the horse performing the same average work, but executing the average pull of 75 lbs. with stream, and 300 against it.

The values required are given for the limits in the following form.

For barges having 113 tons' displacement, and a coefficient $b = 0{\cdot}17$, the results are as follows:—

With the current.			In still water.	Against the current.		
$v_1 = 3{\cdot}0$	2·5	1·0	0	1·0	2·5	3·0
$V = 1{\cdot}79$	1·88	2·2	2·5	2·86	3·66	3·97
$v_2 = 4{\cdot}79$	4·38	3·2	2·5	1·86	1·16	·97
$V_2 =$	5·00	3·5	2·5	1·5	0	—0·5

ere v_1 is the velocity of the current, whether favourable or
rse.

V is the velocity of the barge through the water.

v_2 is the speed of the horse.

V_3 is the velocity through the water for the case in
ch a steam-barge is used, and is given to illustrate the com-
on.

5.—ON VARIOUS HYDRODYNAMIC FORMULÆ.

he results of the various formulæ given for determining dis-
rges, according to various authors, vary very greatly; and it is
ce interesting to examine them in a tabulated form in comparison
h measured discharges.

he following data of comparison are given by Mr. David Steven-
, and by Captains Humphreys and Abbot; they apply to four
s of river discharge, from a small stream up to the Mis-
ippi: thus including all limits within which such formulæ are
uired.

1. For a small stream of 24 cubic feet per second. Mr. David
venson made careful measurements, and velocity observations,
d compared the measured results with the results of formulæ,
us:

1. Measured discharge	24·22
2. By Dubuat's formula	32·50
3. By Robinson's formula	36·90
4. By Ellet's formula	46·40
5. By Beardmore's tables	38·92
6. By Downing's formula, coefficient 1·00	41·23
7. By Leslie's formula, coefficient ·68	28·04

2. For a river of 2424 cubic feet per second. Mr. David
venson and Dr. Anderson made velocity observations on the
y, at Perth, and the comparisons are thus:

1. Measured discharge	2423
2. By Dubuat's formula	2987

3. By Robinson's formula ... ...
4. By Ellet's formula ... ...
5. By Beardmore's tabular formula ... ...
6. By Downing's formula, coefficient, 1·00 ... ...
7. By Leslie's formula, coefficient, ·68 ... ...

It is unfortunate that in these two cases the hydraulic which would enable us to extend the comparison to other for are not given.

3. For a large river of 31 364 cubic feet per second; the d the Great Nevka, measured by Mr. Destrem, were as follows:

Area of section 15 554 width 881
Discharge 31 364 perimeter 898
Mean velocity 2·0486 maximum depth 21
Slope . . ·000 014 87

The following are the results due to these data calculated by v formulæ for mean velocity of discharge:

1. Measured discharge 3
2. Young's coefficient 2
3. Eytelwein's coefficient 2
4. Downing's coefficient... 2
5. Dubuat's formula 1
6. Girard's formula 2
7. De Prony's canal formula 2
8. Young's formula 1
9. Dupuit's formula 2
10. St. Venant's formula 21
11. Ellet's formula 1
12. Mississippi new formula 8

4. For a very large river, the Mississippi at Carrolton measured data at high water in 1851, were,

Area of section 193 968 width 2653
Discharge 1 149 948 perimeter 2693
Mean velocity 5·9288 maximum depth 136
Slope ·000 020 51;

he corresponding results, which are kept in terms of mean ity to reduce figures, were,

Measured	5·9288
Young's coefficient	3·2400
Eytelwein's coefficient	3·5898
Downing's coefficient	3·8434
Dubuat's formula	2·7468
Girard's formula	4·8148
De Prony's canal formula	3·7271
Young's formula	3·2741
Dupuit's formula	4·3752
St. Venant's formula	3·4907
Ellet's formula	3·0451
Mississippi new formula	5·8903

careful examination of these results in four cases of rivers not fail to be instructive; but before entering into comment the discrepancies and their peculiarities, let us also examine the owing list of total discrepancies of mean velocity in thirty cases rivers, streams, and canals of all sizes given by Captains Hum-eys and Abbot in the Mississippi Report, which would, no ubt, be more instructive were the cases classified as to size.

The total discrepancies are:

1. Measured mean velocity of discharge discrepancy	0·
2. Young's coefficient	32·9420
3. Eytelwein's coefficient	28·4411
4. Downing's coefficient	26·6988
5. Dubuat's formula	40·4417
6. Girard's formula	37·4472
7. De Prony's canal formula	28·0905
8. Young's formula	33·3834
9. Dupuit's formula	25·1488
10. St. Venant's formula	30·6619
11. Ellet's formula	45·3547
12. Mississippi new formula	6·3920

From this last table of derepancies it appears that the Mississippi new formula is by far the most correct, and after it th formulæ of Dupuit and Downing, while the two worst are th formulæ of Ellet and Dubuat; but then it must be remembere that the greater number of these thirty cases are those of large an very large rivers.

In the fourth of the previous cases, a very large river th Mississippi new formula is by far the most correct, and then com in order of correctness, Dupuit, Girard, and Downing, while Ell and Dubuat are again the worst. In the third case, Downing i most correct, then Dupuit, afterwards the Mississippi new formul Ellet and Dubuat again the worst. In the second case Ellet an Dubuat remain the worst, and the best are Robinson, Beardmor and Downing. In the first case Leslie and Dubuat are best, an Downing worst.

It will be understood that the formula mentioned as Downing being more familar to many under that name, is really that d'Aubuisson, but applied to English measures.

The inevitable conclusion from all these comparisons is that n one of these formulæ is correctly applicable to rivers of differen sizes, nor holds its own equally as regards correctness throughout For the few and special cases in which the discharge of an extremel large river is required, the Mississippi new formula would necessarily be used, in spite of its form being rather unwieldy; and i the same way Dupuit's formula for a large river. But for ordinar general purposes the thing that the practical hydraulic enginee requires is a formula tolerably well suited to all cases and of simple form, so as to admit of easy rapid calculation. The mos simple type of formula is that of Downing or d'Aubuisson, whic gives for mean velocity of discharge

$$V = 100\ (RS)^{\frac{1}{2}}$$

where R = mean hydraulic radius
and S = mean hydraulic slope;

and this, too, is the formula shown to have been generally the most correct throughout all the comparisons and discrepancies, failing only in the very smallest streams, and evidently worse according a the stream or discharge is less; this then is evidently the bes *formula* for general purposes, and simply requires modification b *experimental* coefficients to answer all ordinary requirements.

The formulæ of Young, Eytelwein, Beardmore, Stevenson, and slie, all belong to this type, merely using other numerical coeffi- nts instead of 100.

Putting Downing's formula into the general form

$$V = c \times 100\,(R\,S)^{\frac{1}{2}}$$

where $c = 1$ according to Downing,

e values of c, according to the other formulæ of the same type are ns :

	$c =$
Young, for large streams	·843
Neville, rivers, velocity $<$ 1·5 feet	·923
,, ,, ,, $>$ 1·5 feet	·933
Eytelwein, generally	·934
Beardmore, open channels	·942
Stevenson, for rivers of 30 cubic feet	·690
,, ,, 2500 cubic feet	·960
Leslie, small streams	·688
,, large streams	1·
Downing Taylor D'Aubuisson — for open channels	1·

From the comparison of the results of the formulæ containing hese coefficients, we may then tabulate values of c that will be actically correct, when suitably applied into the general formula. The comparisons before mentioned show that Downing's coeffi- ient 1·00 gives too small results in cases when the area exceeds 7000 square feet, with a mean velocity of 2·5 ft., or a discharge of 17 500 cubic feet per second, and too large results for cases of smaller data; that the Eytelwein coefficient ·934 in the same way is too small above and too large below discharges of about 2000 cubic feet per second; and the Young coefficient ·843 is incorrect for everything above 900 cubic feet per second; also that for petty streams of 25 cubic feet per second, a coefficient of about ·600 is tolerably correct.

It is evident then that with a very large number of cases of carefully measured discharge, this principle of determining practical efficients in relation to approximate volume or velocity might be carried out to further exactness; allowances for irregularities, lateral bends, and so forth, being made independently of coefficient, as would be done in any case.

Some tabulated values of c, determined in this way, suitab canals in earth in good order, are given in Chapter I., page 30

To apply this same principle to discharges through pipes, ta the same general formula,

$$V = c \times 100\,(R\,S)^{\frac{1}{2}}.$$

As this formula becomes more convenient in practice in term the diameter of the pipe (d), it becomes for full cylindrical p and tubes, where $R = \frac{d}{4}$

$$V = c \times 50\,(d\,S)^{\frac{1}{2}}.$$

And again as the actual discharge is the quantity most often wa this is

$$Q = A v = c \times {\cdot}7854\,d^2 \times 50\,(S\,d)^{\frac{1}{2}}.$$
$$= c \times 39{\cdot}27\,(S\,d^5)^{\frac{1}{2}};$$

and transposing this

$$d = \frac{1}{c^{\frac{2}{5}}}\,{\cdot}23\left(\frac{Q^2}{S}\right)^{\frac{1}{5}}$$

Taking, then, an example in order to compare the result the various formulæ,

Let $Q = 18{\cdot}57$ cubic feet per second
$S = 1$ in 1276,

and the results then are for diameter:

1. By Dubuat's formula 39
2. By Neville coefficient ·228 36
3. By the above formula, coefficient 0·23 37
4. Young's modification of Eytelwein 37
5. Beardmore, coefficient ·235 37
6. Hawksley (in Box's tables) 39
7. De Prony and Darcy 47
8. De Prony's modification of Dubuat 48
9. Gerney 48

Besides these, there are very many authors that would g results for diameter very much below that of Young; it appears t that none of these formulæ apply equally well to both high and *velocities* of discharge, although it is unfortunate that a sufficie ge number of data are not forthcoming to determine correctly limits at which it would be advisable to change the coefficient.

The above comparisons while showing the merits of the various mulæ in certain cases, also point to the very evident conclusion, t a variable coefficient of discharge is necessary both for rivers, annels, and pipes; and that it must be suitable both to the ensions and the conditions of each particular case. The best de now known of doing this is that of Mr. Kutter of Bern, which applied to English measures in Chapter I. of this Manual: the ues of coefficients being also given in the Working Tables.

6.—IRRIGATION FROM WELLS IN INDIA.

There is unfortunately a large number of Indian officials that lieve that irrigation from wells in India is more profitable than igation from canals conveying the water of rivers and delivering on the surface of the land by the aid of gravity alone; they nerally are men not likely to be persuaded to the contrary by gineers, however good their reasoning might be; and, unfor- nately, engineers are not always provided with facts and figures. these latter, therefore, the following data may be of service; ey were drawn up as applicable to the years 1855 and 1870; e former by Captain Baird Smith, the latter by the author.

omparison of Irrigation by Wells with that by Canals for a District of 1 500 000 *acres in Northern India in* 1855.

By wells—Capital :—	
Wells costing £20 each, for 10 acres ...	£3 000 000
Machinery, etc. (and bullocks?)	1 000 000
	£4 000 000
Annual expenses :—	
300 000 men at £3 a year	£900 000
1 200 000 bullocks at £1$\frac{1}{5}$ a year ...	1 440 000
10 per cent. interest on capital	400 000
	£2 740 000

By canals :—

(assuming the data of the Eastern Jumna Canal).

Capital :—

1 500 000 acres at 5*s.* an acre	£375 000

Annual expenses :—

Water rent at 2*s.* an acre	£150 000
Watercourse repairs at 7*d.* an acre ...	43 750
Labour at £2 8*s.* per annum	72 000
10 per cent. interest on capital	37 500
	£203 250

Comparison in favour of Canals.

Capital 1 to 9; annual expenses 1 to 13.
Saving effected annually 2½ millions.

Comparison of Irrigation by Wells with that by Canals in Northern India, for 1870.

Data.—The Eastern Jumna Canals in 1864–65 had cost 15 per acre irrigated; the Western Jumna, up to 1863–64, had co 12*s.*: hence assuming 20*s.* for a less favourable canal.

By canal :—

	Per Acre. £	*s.*	*d.*
Capital expended on a developed canal should not exceed	1	0	0
Return levied by water-rate, dues, and increase of land assessment	0	15	0
Working expenses	0	5	0
Net profit 50 per cent.	0	10	0

By wells :—

	Per Acre. £	*s.*	*d.*
Capital expended on a well 10 feet deep with machinery, &c., to irrigate 10 acres at a cost of £30, gives a cost per acre	3	0	0
Working expenses, including interest on prime cost	1	0	0

Comparison in favour of Canals.

Capital 1 to 3; annual expenses 1 to 4.

Saving effected annually on a district of 1 500 000 acres, ⅛ millions.

The profit of 50 per cent. net, allowed in the last comparison, has already been exceeded on the Eastern Jumna Canals; nor is that nearly so high a profit as might have been effected had the works been carried out steadily, continuously, and by experienced engineers, under arrangements that would have caused or forced the landholders at once to utilize all the water, or sell their lands to others that would do so.

Other important data in connection with irrigation canals are, the saving effected by doing away with remission of land assessment in famine years, and the value of the produce and cattle saved in years of drought; the indirect advantages to the country and the Government, resulting from increase of produce and of population, are innumerable. Well irrigation, on the contrary, fails at the time when it is most wanted, the ordinary wells, being shallow, drying up in years of drought.

In the Hydraulic Statistics are some data having reference to irrigation from wells in different parts of India.

7.—THE WATERING OF LAND.

The following is the usual mode of classifying crops with regard to their special treatment under irrigation. 1. Grass meadows, or natural meadows of gramineæ. 2. Dry grain crops or cereals. 3. Leguminous crops. 4. Root crops. 5. Those specially requiring more water: rice, indigo, tobacco, sugar, bamboo, water-nuts. 6. Garden or fruit crops. 7. New plantations, and trees.

Peculiarities of climate, soil, and water will generally affect the amount of water required for irrigation probably more than than the species of crop. In England meadows of grass land, or Italian rye-grass, are those that generally profit most from irrigation. The usual plan is to keep the land flooded to a depth of two inches during the months of October, November, December and January, for twenty days at a time, and then to let the water drain off from it for five days, before putting it again under water. In frosty weather, however, the field should always remain flooded. In February and March the fields are flooded for eight days at a time at night only; at the end of March the land is left dry; and in May the grass-crop is cut. Irrigating fields in England in the hot weather is liable to produce rot in sheep, but does not harm cattle.

There are two methods of laying out the courses or channels in English fields:

1. The bedwork system, applicable to flat land.
2. The catchwater system, applicable to steeper country.

According to the former, the land is made into a series of very flat ridges, having a general direction nearly at right angles to the channel of supply, and being never more than 70 yards long and about 40 feet wide, the inclination of the ridge itself having a fall of about 1 in 500, and the inclinations of the sides of the flat ridges varying with the retentive power of the soil, from 1 in 100 to 1 in 1000; the crown of the ridges is not necessarily, therefore, in the middle of the breadth of the base of the ridge. The feeding and drainage channels are generally from 20 inches wide at their junctions to 12 inches at their ends.

The catchwater system used in Devonshire and Somersetshire consists of a series of ridges made across the general course of the water, which hold the water up, and retain it over successive long strips, the water passing slowly round the end of one ridge to the lower land above the next ridge, and so on. This is necessarily far cheaper than the other system—about half, and can be carried out at the cost of about five pounds an acre.

Throughout the world generally, there may be said to be only four methods of distributing water on or throughout surfaces, of which all others are mere modifications. In all cases it is best that the land should have one general slope throughout, the irrigation channel running along the head of this slope, the main catchment drain along the bottom.

The first method is that to which the English bedwork system

ongs, the field being prepared in furrows and ridges alternately
m the head to the foot of the slope, either in the direction of
fall or making an angle with it, according as the quality of
soil and the general slope of the land may require; these
furrows, being from 10 feet to 50 feet wide and only a few
hes in depth, receive the water from the irrigating channel,
ich will then cover the land nearly up to the crests of the
ges, or in fact entirely if need be.

The second method is very similar to the first, but the water,
stead of flowing in the furrows, runs in little trenches cut along
e crests of the ridges, overflows the sides, waters the slopes, and
rains off in the furrows down to the main catchment drain. The
idges used in this system are generally wider than those of the
st system, and have a greater lateral inclination.

The third or commonest method for applying water on a small
le is to distribute the water in little trenches around small
uares and rectangles of land, allowing it to permeate throughout
surface enclosed, which must be very nearly level with the
ter in the trenches.

The fourth method, most commonly adopted in Spain, Portugal,
India, in cases where much water is required to remain on the
d for some time, as for rice-crops, or many grain and other
ps in their early stages, that could not thrive on hard baked
, consists in levelling the land into a number of nearly flat
ares and rectangles, divided from each other by small ridges or
rf mud walls, to hold the water on them. The number of rect-
gles depends on the fall of the ground; the water is allowed to
in at some corner or temporary break, and flow out in the same
on to the next rectangle when it has remained sufficiently long.

s to soil :—For the surface, the most permeable is best, being
t easily warmed, and allowing the water to arrive at the roots
he grass most quickly; a retentive surface-soil causes evapo-
ion, and cools the land, which is generally a disadvantage,
ugh not so under some circumstances;—a subsoil of clay, being
ative, is an advantage in very dry climates, as it economises
er. In hot climates the soil is of inferior importance to the
ity of the silt transported and deposited.

all the water as distributed, a mode more likely to be adopted at present, now that modules are less expensive and more effective than formerly.

2. By area of land irrigated, or by crop.

This has the following disadvantages; the land to be irrigated is always varying in amount, and this cannot be watched in detail continually, nor can the landowners be trusted to state truthfully the amount of acreage over which water has been distributed. The crop can also be varied, so as to use more or less water, and the payment by crop also would be useless against cheating. Again, in a good rainy season the cultivator might try under these circumstances to do without the canal water, thus causing the water-rate to be precarious.

3. Water distribution by rotation.

An irrigating channel of fixed dimension, giving a constant fixed discharge, passes through the lands of several proprietors; a period of rotation is fixed for this channel, from 6 to 16 days according to the crops, the former for rice and the latter for meadow land, as, for instance, in Italy. Each landowner can then have the whole volume of the channel turned on to his land once in the total period of rotation for a certain number of hours, as from two to forty or fifty according to the amount of land he owns.

For example. Let ten days be the period of rotation, and let him require twelve hours' supply once in that period. His name is placed on the list, say sixth, and he gets his supply turned on at a fixed hour and turned off at a fixed hour also. If the channel gives twenty cubic feet per second, his amount of water is equivalent to a continuous discharge of $\frac{20 \times 12}{240} = 1$ cubic per second. In this way intermittent supplies admit of mutual comparison.

Last with regard to the cultivators themselves:—Whether on the Continent, or in England, the farmer is generally a grumbler under any state of affairs. In India the cultivator invariably complains, although his assessment is very small by comparison with the local circumstances; if he grow two very moderately good crops in the year, it would only amount to about two and a half per cent. per annum on the value of the produce, and he can therefore well afford to pay large water-rates, especially since both the yield and the number of crops produced on irrigated land is doubled, and the highest water-rate is small in comparison with the expense of making wells and raising the same amount of water by animal power throughout the year; he enjoys also the advantage of living under a government that remits the land assessment, and distributes food gratis in years of famine, while not demanding more assessment in years of plenty. If the water-rate is in some just proportion to the increase of produce and saving of expense resulting from the irrigation, it matters not how large per acre the rate may appear to be. If the irrigation is applied to suitable land in such a way that the natural drainage of the country is not interfered with, there can be no detriment to the health of the cultivator; this can, however, be rarely carried to perfection in actual fact. To this it can be replied, that the population will thrive on the whole and increase largely, which may be considered as a set-off on that account, and that landowners who prefer going away can always do so and part with their land at a premium; land always commanding a ready sale. A compulsory water-rate on land that is under water command cannot be considered a hardship by any one that considers the subject in a fair, unprejudiced manner; the privilege of being able to obtain water should be paid for, and since the same principle has always been applied to town supply of water, for which every inhabitant has to pay whether he uses it or not, there is no reason for leaving the payments of water-rate in the country to be optional. Whether both the landowner and the occupier should pay separately for the advantages they both receive is a point dependent on the local tenure of land; under ordinary circumstances they doubtless should do so, the occupier being benefited by increase of produce, the landowner by increase of rent; but in any case the whole of the advantages should be paid for.

8.—CANAL FALLS.

That a fall of water at the headworks, or at any part of a canal, should be allowed to remain unutilized, appears, in these days of expensive fuel and costly motive power, to be a very painful waste of a valuable advantage. One's natural tendency is to devise means and ways of using everything, and to imagine that there could hardly exist circumstances under which it would be necessary to arrange for the destruction of the power and velocity generated by a fall of water. Grinding corn, pressing sugar, or extracting oil, are requirements even in semi-barbarous countries, by which such motive power could be easily utilized, even if it were available for only four months in the year. In spite of this, however, it seems rather frequently to occur, that in distant countries the engineer has to devise means for destroying the effect of a fall of water; this occurs, generally, either at the headworks of a canal, where the water entering the canal in flood seasons has a great head of pressure, or at certain points in a canal where, owing to the inclination of the country being steeper than that due to a convenient velocity of canal current, it has been found necessary to concentrate the superabundant fall: the Ganges Canal and the Bari Doab Canals have many such examples. In either case, as the fall is independent of navigation of any sort, which has to be conducted in a special channel of détour, the problem is one of economy. The natural means would be to break up the force of the water by both lateral and vertical breaks and angular obstacles, and to oppose the remains of the velocity by a pierced breakwater, beyond which the water would issue with so small a current as not to be able to cause any damage to the bed and sides of the canal, or to cause any prejudicial effect to navigation.

The breakwater, involving an enlargement of the width of the channel, and, if a rock foundation be not available, requiring artificial and carefully made foundations carried to some depth, is necessarily expensive, and is hence generally dispensed with, except under favourable circumstances.

The fall itself is generally a modification of one of the three following types:—

1. A uniform, or a broken general incline.
2. A vertical fall with gratings.
3. A vertical fall with a water-cushion.

The most primitive mode of managing such falls of water was to conduct it down an incline, made as gradual as possible, and break up the velocity by a series of steps.

A long reach of rocky bed offers a convenient opportunity for construction, which could be hewn in the solid rock. In cases, where it would require building on artificial foundathe expense would be very great; and, even if the incline so made that the resulting velocity were not high, the edges of reads of the steps, even in good stonework, would soon wear, he maintenance of the fall would also become an important of expense. Apart from these objections also, this type is tisfactory. Although the treads of the steps may be set with rrect reverse inclination, so as to oppose more directly the ned direction of motion of the momentum of the water; and, ough a further improvement may be made in giving a more siderable reverse inclination to the treads, and by allowing a e proportion of the water to run off laterally and wind down steps; yet under all circumstances the inherent defects re- in; the steps cannot accommodate themselves to the variation the quantity of water passing down the fall; if the steps are ll, they fail to receive effectively the over-falling water when the ount increases, and become then comparatively valueless; if steps are very large, the rise and tread of each step causes the ocity acquired from each step, which it must be remembered reases in the ratio of the square of the height of the step, to very much increased, and to become very destructive to the nework.

The next improvement on the inclined type of fall is the ogival ll used on the canals of Northern India; in this the general slope descent from the head to the foot of the double curve is from e to six to one in nine; the upper one-third of the slope being e chord of the upper or convex curve, which is tangential to the rface of the water in the upper reach; and the lower two-thirds the slope being the chord of the concave curve, which is tangential the convex curve above, and tangential to the horizontal line at its wer extremity. The height and length of the fall applicable to y special case is determined by equating the discharge of the en channel above with the discharge over a weir. The principle hich this form of construction asserts is that the water at the foot the descent, being deprived of all vertical action and delivered rizontally, will not cause any damage to the bed of the channel the lower reach.

In canals where it is required that the discharge should r perfectly uniform and unaffected by its fall down the weir or incli these ogival falls must necessarily have their sills raised above level of the channel-bed of the upper reach; as would also a f uniform slope.

Curves on more carefully eliminated principles have also b tried with the object of effecting some improvement, but the adv tages resulting appear comparatively small. These curves gene effect no doubt some saving of masonry in comparison with that a single uniform slope, and probably deliver the water with destructive result than the latter; they are, however, still expens and the action of the water delivered is rather concentrated, hence destructive. An attempt at economy on such falls has b made by narrowing the fall, and thus diminishing the amoun masonry; but the results, caused by the increase of action well as irregularity of effect of the water, require greater expen ture in repair; they present also the additional disadvantag that during repair the whole fall instead of a part has to stopped.

In the above cases of inclined falls it is supposed that it has be found convenient to concentrate the fall in a comparatively sh length; in other cases, where it is spread over a long reach, it usual to attempt to annihilate the velocity resulting at the foot the incline by introducing a reach of canal having a reverse slop and in cases where a greater length still can be allowed for th incline, to break it up into portions of descent, each followed by portion with a reverse slope and then a short horizontal lengt thus opposing the accelerating effect in detail without allowing i results to accumulate. In such work the bed of the channel m necessarily be paved; if the velocity do not exceed 10 feet or 1 feet per second, large rough convex boulders, laid dry, form the most suitable paving; and even up to 15 feet per second the same method may be adopted if very large boulders alone are used; beyond that velocity the boulder work requires packing with shingle and pebbles, and grouting with good hydraulic mortar.

While the above arrangements may destroy a great deal of the velocity, there is perhaps almost always a certain amount of it still *remaining* at the foot of the incline, and should the channel at this e happen to be in soft soil, further arrangements, tail-walls, hwood spurs, or piles, are also necessary.

he Bari Doab Canal tail-walls offer an example illustrating such se, the arrangement being generally as follows: At the foot of incline the bed of the channel is made horizontal for some ance, and the banks are then splayed outwards in a curved n until the top width of the channel at water level is one-half er than before: this, giving additional water way, reduces the ocity; the channel is then narrowed to nearly its normal width walls of dry boulders on each side, which project into the stream an inclination of 1 to 5, and slope longitudinally with a fall of n 20 from their commencement, where their height is up to full ply-level, down to the level of the bed: these are, of course, ally submerged at full supply, and produce the effect of contrating and directing the current to the middle of the channel. e objections raised to these tail-walls as employed on the Bari ab Canal, is that they do not appear to answer their purposes ficiently completely, and it is supposed that by giving the whole rangement, both the enlargement and the reduction of section, a eater length, it would fully answer all purposes; this, however, ould add greatly to the expense.

Vertical falls with gratings. This is, perhaps, the most onomic and convenient mode of dealing with a canal-fall. The l of the fall is not raised above the bed of the upper channel d the whole section of passage is hence unimpeded by reducon; the grating, which may be placed at any slope from 1 in 3 1 in 10, presents a large perforated surface to the action of e water, thus keeping the upper water up to its proper level, d distributing the effect of the falling water passing through on a long portion of the bed, diminishes the action to such an tent as to render it harmless. The gratings are supported on oss bearers, which again rest on masonry piers or iron stanchions, ected at about 10 feet intervals along the edge of the fall or ir. The higher a fall of this description is, the more truly the ter falls and the more manageable it is. These gratings require aring occasionally, and hence necessitate the attendance of a n; but as frequently there is a lockman to attend to the neighring lock, for the *navigation passage* near the fall, there is no additional expense incurred on this account, as one ma attend to both. This type of fall admits of comparatively variation in design.

Vertical falls with water-cushions.—This is the form gen adopted by nature in discharging water down a fall; the acti the water scours for itself a basin, which fills and forms a n water-cushion, the scour continuing until an equilibrium is lished between the force of the descending water and the r ance offered by the depth of water in the basin. The fall has a tendency to approximate to the vertical, the force of and spray from the falling water making it slightly overhan and in some cases even causing a retrogression of fall, and cidently also a retrogression of water-cushion, thus giving elongated form; the scoured silt, or débris, is deposited in th of the stream lower down.

The most natural mode of designing a vertical fall with cushion for a canal would perhaps depend on a considerat what sort of fall nature would make for herself under the s circumstances and conditions of the case, and what improve or modifications of that would be necessary. The objectio allowing nature to make her own fall and water-cushio these:—first, it requires time, and this, in some, though n all cases, is an objection in itself; second, any want of geneity of the soil or rock would result in an irregular fo basin, which might become almost unmanageable; third scour and silt deposited in the channel below would be a s injury to it; fourthly, the retrogression of the fall might tually undermine the weir or dam, and cause its entire destru But this latter objection might be very easily counteracted b tective measures.

In cases, then, where these four objections can be ren or are unimportant in result, there is no reason why a n or a slightly modified natural fall should not be adopted. the soil is firm or of homogeneous rock, a great deal o objection disappears, a certain amount of excavation and ming can then be so made as to aid in the natural action lateral encroachment may be easily provided against; a tole regular basin can then be economically made.

As to the form of basin best suited for a water-cushion, the adth in plan should be rather wider than the extreme breadth the falling water, as the wind may bear the latter considerably one side; the length, again, will probably vary from 1½ to 5 es the breadth, although it would hardly be advisable to make quite rectangular in form, as the corners would be filled with less water; the pear shape, therefore, is perhaps the best, and certainly that most generally met with under natural conditions homogeneity of soil. There would probably be no advantage, en if it were economic, to make the basin very long; the full extreme depth may be terminated by a reverse slope at once, deflected velocity thus obtained producing a greater degree of llness than the passive effect of a longer continued full depth.

The main point, however, is to determine what depth of water necessary in a water-cushion. The velocity of delivery is evintly dependent on the depth on the weir sill or fall above, and height of fall down to the surface water in the basin: the sistance is the depth of water in the basin, and the quality of material of which its bottom is composed. If, then, the depth calculated by equating the forces for a depth producing equilirium just clear of the bottom, we obtain an expression, involving so an assumption that the bottom is perfectly indestructible. seems, therefore, impossible at present to determine absolutely actual depth necessary; and hence the practice is to assume approximate calculated depth, and see how this answers its rpose, altering or adding afterwards until it appears to be tisfactory.

The formula generally used for this purpose on the canals of orthern India is—

$$d = 1\cdot5 \sqrt{h_1} \times \sqrt[3]{h_2}$$

d = the depth of water in the basin;
h_1 = the total height of fall, including h_2;
h_2 = the depth or head on the weir sill

his is probably very limited in its range of application; for, in plying it to the well-known case of the projected Masur reservoir m, designed by the engineers of the Madras Irrigation Comny, it yields results very small in comparison to that allowed by engineers: thus, for values of $h_1 = 48\cdot5$ and $h_2 = 6$ feet, the calculated value of d, suitable to a brick bottom, is about 18 fee while the engineers have allowed for a hard rock bottom a dept of water-cushion of 33 feet in this instance.

In a second instance of the same case, the formula gives f values of $h_1 = 16{\cdot}81$, $h_2 = 8{\cdot}56$, $d = 12{\cdot}54$, which is very muc less than that allowed, 16·19 feet, also in hard rock.

Major Mullins, the Consulting Engineer to the Madras Irrig tion Company, when commenting on these cases in the Proceed ings of the P. W. D., for April, 1868, refers also to a well-know natural fall as an illustration of the insufficiency of the abov formula. The Rajah Fall at Gairsappa, with values of $h_1 = 8{\cdot}2$ and $h_2 = 15$ feet, would, according to that formula, require a dept of water-cushion of only 108 feet for brickwork, or 72 for ston a depth nearly a half less than the actual depth, 130 feet.

In a smaller natural case, in hills in Berar, coming under th observation of the author, for values $h_1 = 26$ and $h_2 = 1$, th depth, according to the above formulæ, would be for a brickwork bottom 7·65 feet, and for stone 5·6 feet; whereas, in the soundest of basalt, the actual depth was as much as 8 feet, or more than a quarter more than that calculated.

It would; therefore, appear that the above formula, apart from its varied coefficients for brickwork and stone, is generally defective, and that, until a very much wider range of experiments and observations is made, it would be more advisable to approximate to such depths as are obtained under natural conditions, than to follow any formula for determining the depth of a basin serving as a water-cushion.

In practice it would rarely be necessary to construct a water-cushion of very great depth, the fall, if over a weir, being generally easily broken into three or four portions, and it being advantageous to do so, as the catch channels are convenient for affording a supply at various levels; probably, therefore, the above-mentioned case of 43·5 feet of artificial fall may be considered as the extreme for which a water-cushion would be required. In the future, too, the waste of such a large amount of useful motive power will be deemed a barbarism, an additional reason that there is not much probability of the above case being exceeded.

9.—THE USUAL THICKNESS OF WATER-PIPES.

The thickness of a water-pipe is a matter depending on prac- al considerations, being comparatively little affected by the oretical determination of what it should be in order to resist pressure brought on it; and is, like a very large number of so-called calculations of the engineer, made almost entirely pendent on prescribed custom. The following notes on the mulæ in vogue are, hence, not given so much with the object elucidating the principles as that the formulæ themselves, value- s as they seem, should be available for reference.

The largest scale on which a water-pipe to resist extreme internal essure is made is that of the cylinders of hydraulic presses: in ese the extreme working pressure is limited to 4 tons per square ch, the extreme permanent strain allowed in actual working ing only one half of that; and the thickness of the cylinder or pe is determined by the formula of Barlow—

$$t = \frac{r.\ P}{C - P};$$

here t and r are the thickness and internal radius of the cylinder pipe,

C is the cohesive strength of the material, and

P is the internal pressure, both being in tons:

he general principle asserted in this mode of calculation being hat the strain on the material is greatest at the internal surface, nd less beyond, the extension varying with the square of the istance from the centre.

An example of the application of this formula, to a 10-inch st-iron water-pipe, is given in Box's "Hydraulics," the results of hich are as follows:—

Assuming the cohesive strength of cast iron to be 7 tons per quare inch breaking weight; the extension E, on the inside ring the moment of rupture, for a length = 1,

$$E = \cdot 000\,165\ W + \cdot 000\,010\,3\ W^2 \times L = \cdot 001\,659\,7;$$

nd the extension at any distance from the centre is in the ratio the square of that distance to that of the inside ring.

The strain, at any distance from the centre, is then obtain from the extension by the formula—

$$W = \sqrt{\left(\frac{E}{\cdot 000\ 010\ 3 \times L} + 64{\cdot}16\right)} - 8{\cdot}01$$

and the mean strain on each theoretical concentric ring of me is the average between that at its external and its internal circum ference; the bursting pressure has then the same ratio to th mean strain as the thickness of the pipe has to its radius; an tabulating these for a 10-inch cast-iron pipe, they are:—

Thickness of Metal.	Strain on the Metal.			Bursting Pressure.
	Max.	Min.	Mean.	
1″	7.0	5·26	6·130	1·226
2	7·0	4·09	5·402	2·161
3	7·0	3·26	4·827	2 896
4	7·0	2·65	4·359	3·485
5	7·0	2·20	3 972	3·972
6	7·0	1·85	3·647	4·337
7	7·0	1·60	3·373	4·722
8	7·0	1·37	3·137	5·019
9	7·0	1·19	2·931	5·275
10	7·0	1·05	2·749	5·499

The practical empirical rule, however, that is given by Box for the thicknesses of water-pipes is—

$$t = \left(\sqrt{\frac{d}{10}} + 0{\cdot}15\right) + \left(\frac{H\,d}{25\,000}\right);$$

where H is the head of pressure, and *d* is the diameter of the pipe, and it is according to this, that his table given in the Appendix of Miscellaneous Tables is calculated.

The theoretical mode of arriving at the thickness of a water-pipe is, therefore, about the most unsatisfactory of processes; and it would probably be useless to enlarge on the topic. In actual practice, the dimensions of cast-iron water-pipes are about those given in Box's table; or have a thickness of one-fifth the square root of the diameter, and a little more to allow for defects in casting, and inexactitude of bore. The dimensions of the details of the sockets are also given in the second part of Box's table, and are very convenient for reference.

Flanged pipes being now so rarely used, excepting for temporary purposes, the details of their usual dimensions and weights, given by Box, are omitted in the table given.

While in the case of cast-iron pipes of all sorts, there has always been a tendency to theorise, and to base a thickness on the laws of pressure, and extension of material; in stoneware pipes, this has been almost entirely disregarded, and a thickness is generally given them that is established entirely on practice or usual custom, and often varies according to the caprice of the potter or manufacturer. This is generally accounted for by saying that earthenware or stoneware is a very variable material as regards strength, while cast iron is homogeneous, and is very much alike in substance: a little reflection, however, will show that this is hardly a sufficient reason. Carefully made stoneware, after a very careful selection, may be, and often is, exceedingly equable, while the variety of qualities of cast iron, —more especially since its high price has brought such a large amount of very inferior material into use,—is now very marked; some cast iron being known occasionally to fall to pieces from its own weight. In spite of this, the manufacturers of stoneware pipes still consider them as unsuited to the discharge of water under pressure, or for drainage in cases where the outlet is liable to be stopped: and although they can make pipes that will easily bear a head of 40 feet, yet do not recommend them, alleging that the joints cannot be made to stand any pressure at all. There is, however, no reason to doubt that under skilled superintendence and management, stoneware and fire-clay pipes, as well as their joints, may be well enough made to serve most efficiently for the distribution and drainage of water under low heads, and that a considerable saving of expense may be effected by dispensing with iron in such cases.

10.—INDIAN HYDRAULIC CONTRIVANCES.

In India a large variety of mechanical contrivances of a very simple nature are commonly used for raising water from rivers or wells or out of foundations of bridges, that are generally unknown to the English engineer. His natural tendency would be to use the appliances best known to him, such as a windlass and bucket, a common pump, a lift and force-pump, or a winding-up chain carrying iron vessels; of these the last only is very well known in India in a more simple form, as a chain of pots or leather bags. Pumps are purely European in origin, even a windlass is a comparative rarity; and since such things are not always available it often becomes necessary for him to adopt the native means of raising water and to learn what duty may be expected from them. To aid him, or rather to save him needless trouble in measuring and calculating the duty, the table given in the Appendix to the Working Tables, based upon data originally furnished by M. Lamairesse, of Pondicherry, for Southern India, and in the Roorkee professional papers for Northern India, and in conjunction with others by the author, but modified and put in a form intelligible to the English civil engineer, may be found useful. It merely becomes necessary to give the meaning of a few of the Indian names of the contrivances, and state the mode in which they are used.

Baling is one of the most primitive methods of raising water, but the English mode of filling and emptying a vessel or a bucket is not in vogue among the natives of India. A large flat dish of wood bark rendered water-tight, or leather stiffened by a frame, has two long cords attached to it at opposite sides, the other two ends of the cords being held by two men, who generally prefer sitting down to their work, and together allow the dish to dip in the water, nearly fill itself, and then raise it, send it forward with a swing and let it empty itself above; this can be done with a rapid and continuous swinging motion that is sometimes quite surprising. This method is of course only applicable under certain conditions, such as clearing foundations of water, and such cases as allow of sufficient room for the swinging; the lift is seldom more than 5 ft. though sometimes 7 ft.; but a series of such lifts can be easily adopted.

The beam and bucket, or balance-pole, in its various modifications, is also a favourite contrivance for raising water from wells by hand labour; the lever, at one end of which is hung the water vessel, generally a large earthenware pot, is counter-weighted at e other end so as just to allow the force of one man to raise the ssel when full. The lever is often a beam naturally very thick one end, and requiring only to be carefully hung or supported at e most convenient point for a fulcrum. In Southern India this inciple reaches its fullest development in the picotah; where a ry large long tree, or a very large pair of trees bound together, ecomes the balance-pole, to work which a man walks and runs ckwards and forwards along the heavier arm of the lever, epping off, when necessary, on to a raised stage; for this work pecial men, thoroughly accustomed to it, are absolutely necessary; ne managing the vessel, the other the balancing. The size of hese picotahs is sometimes extremely large, and the lift consequently very high.

The dal or jantu is a contrivance for raising water from 3 ft. to ft. high by means of a wooden gutter moving on a pivot, being a lever, or a double lever of the second order. There are several orms of this contrivance; in the simplest, one end of the single gutter is raised by a man with a cord or lever and cord, until the ater runs out of the other end of the gutter into a trench; in the ouble gutter there is a wooden partition in the gutter immediately bove the pivot, and the water runs out through holes on each ide of it in the bottom of the gutter into the trench; sometimes hese are worked by cords, and sometimes by means of the weight of a man and a counterpoise at the end of a long lever attached.

The mot is an arrangement worked by oxen; it generally onsists in a water vessel made of a complete ox-hide bound on a wooden ring for an opening, raised and lowered by a cord unning over a pulley, and fixed immediately above or projecting over e well: the bullocks going down an inclined plane made for the rpose, when dragging up the water vessel or mot, which has to e dragged to one side on arrival above the mouth of the well and mptied by a man. In Southern India there is an improvement n this which dispenses with the man for emptying; the lower end f the mot tapers out to a considerable length, and has a smaller rd attached to it, which by means of a suitably adjusted catch uses the mot to empty itself on arriving at the proper height.

The contrivance generally called by Anglo-Indians a Persian eel, but more properly a chain of pots, is almost identical with that used in Egypt, Nubia, Syria, Abyssinia, known there as the sakia; its advantage is that it will raise water from any depth by means of sufficient animal power. In India it is generally very much of the following description. Two parallel endless ropes, united to each other by rungs of wood or of rope, pass over a vertical wheel and hang down to below the water surface in the well; earthen or leathern vessels are attached to the rungs, which discharge themselves into a trough through the vertical wheel, which is a double frame-work. Motion is communicated to the axle of this vertical wheel from a vertical shaft of wood that is turned by a pair of bullocks, by means of two wooden wheels working into each other. The upper end of the vertical shaft is kept in position by a very heavy beam or tree which rests also on two supports, generally mud walls, beyond the sweep of the circle in which the oxen walk. The principle of this rather rude but effective contrivance was doubtless the basis of the double iron chains of pots, with brass buckets holding about a gallon each, that were used by the Romans, and hence also the remote ancestor of our modern chains of pots having chains of jointed iron bars, skeleton six-spoked or hexagonal wheels, and buckets or iron casks of the most improved form; or again, somewhat like those used and worked by steam power on the Metropolitan District Railway to clear the line of water.

The true Persian wheel, with which the chain of pots is sometimes confounded, is a wheel with a hollow tyre, and is an inferior contrivance, suitable only to small lifts.

Referring to the table given, the details of which have been reduced and modified in order to show as much as possible what comparison may be drawn in favour of each machine, it will be noticed that the full amount of work done and power exerted is, in the first place, given for all cases, under a theoretical condition that never occurs in practice. In each and all of these machines, a certain amount of work is wasted by leakage, spilling, faulty construction, or inexactness of form, delay for small repairs, and many other such causes. To obtain anything near the truth, therefore, a coefficient of reduction that is purely empirical must be applied. Some of these coefficients are given in the Roorkee professional *papers*, others are obtained from other sources; they may for our es in dealing with such rough machines be applied equally to rk done and the effective power exerted; but as the latter is ncipal object under consideration, the amounts under that nly are reduced. The final quantities, therefore, are more illy useful.

results may not at first sight appear to admit of much rison being made; certain things are, however, plainly in- by them, the most marked one being that all such rough lic contrivances used in small lifts involve a great waste of as well as of water, much intermediate time being lost n the lifts, and that the machine itself, when on a large being more properly made and more carefully worked is re effective. This is shown most on comparing the effective of the North Indian beam and bucket (12) with the rn Indian picotahs (1, 2, 3); in the mots, on the other hand, vantage is on the side of the North Indian, probably from ng an additional man, although it is probably obtained at eat an expense. The chain of pots more exclusively used thern India appears to be, under theoretical conditions, the effective of all these contrivances. The data given are, ie very variable nature of such things, too rough to allow comparison being drawn between such contrivances and the ivilized arrangements; but they may, however, be of use se unacquainted with Indian contrivances when first called leal with them.

[redacted]

HYDRAULIC WORKING TABLES.

ELEVATION.	20°	25°	30°	35°	40°	45°	50°	55°	60°
0	32·0994	32·1108	32·1238	32·1383	32·1536	32·1695	32·1854	32·2008	32·2152
100	32·0991	32·1105	32·1235	32·1380	32·1533	32·1692	32·1851	32·2005	32·2149
200	32·0988	32·1102	32·1232	32·1377	32·1530	32·1689	32·1848	32·2002	32·2146
300	32·0985	32·1099	32·1229	32·1374	32·1528	32·1686	32·1845	32·1998	32·2143
400	32·0982	32·1096	32·1226	32·1371	32·1524	32·1683	32·1842	32·1995	32·2140
500	32·0979	32·1093	32·1223	32·1368	32·1521	32·1680	32·1839	32·1992	32·2137
600	32·0976	32·1090	32·1220	32·1364	32·1518	32·1677	32·1835	32·1989	32·2134
700	32·0973	32·1087	32·1217	32·1361	32·1515	32·1674	32·1832	32·1986	32·2131
800	32·0970	32·1084	32·1214	32·1358	32·1512	32·1671	32·1829	32·1983	32·2128
900	32·0967	32·1081	32·1211	32·1355	32·1509	32·1668	32·1826	32·1980	32·2125
1000	32·0963	32·1077	32·1208	32·1352	32·1506	32·1665	32·1823	32·1977	32·2121
2000	32·0933	32·1047	32·1177	32·1322	32·1473	32·1633	32·1793	32·1947	32·2091
3000	32·0902	32·1017	32·1146	32·1291	32·1442	32·1603	32·1762	32·1916	32·2060
4000	32·0871	32·0986	32·1115	32·1260	32·1411	32·1572	32·1731	32·1885	32·2029
5000	32·0840	32·0955	32·1084	32·1229	32·1382	32·1541	32·1700	32·1854	32·1998

Being an application of the formulæ $g = 32{\cdot}1695\,(1 - {\cdot}00284 \cos 2l)\left(1 - \frac{2e}{r}\right)$

$r = 20887540\,(1 + {\cdot}00164 \cos 2l)$

TABLE II.—Part 1.

Total quantities of water equivalent to a given rainfall.

Rainfall in feet.	Cubic feet per acre.	Cubic feet per square mile.	Rainfall in feet.	Cubic feet per acre.	Cubic feet per square mile.
1·	43 560	27 870 400	(12″) 1·	43 560	27 878 400
·9	39 204	25 090 560	(11″) ·917	39 900	25 555 200
·8	34 848	22 302 720	(10″) ·833	36 300	23 232 000
·7	30 492	19 514 880	(9″) ·750	32 670	20 908 800
·6	26 136	16 727 040	(8″) ·666	29 040	18 505 600
·5	21 780	13 939 200	(7″) ·583	25 410	16 262 400
·4	17 424	11 151 360	(6″) ·5	21 780	13 939 200
·3	13 068	8 363 520	(5″) ·417	18 150	11 616 000
·2	8 712	5 575 680	(4″) ·333	14 520	9 252 800
·1	4 356	2 787 840	(3″) ·250	10 890	6 969 600
			(2″) ·166	7 260	4 646 400
			(1″) ·083	3 630	2 323 200
·09	3 920	2 509 056			
·08	3 485	2 230 272			
·07	3 049	1 951 488			
·06	2 614	1 672 704			
·05	2 178	1 393 920			
·04	1 742	1 115 136			
·03	1 307	836 352			
·02	871	557 568			
·01	436	278 784			

For decimals of an inch of rainfall remove the point in the corresponding quantities.

N.B.—One square mile = 640 acres = 27,878,400 square feet.

[redacted]

TABLE II.—Part 2.

arges in cubic feet per second throughout the year, equivalent to given annual rainfall over one square mile of catchment area.

nual fall in et.	Discharges in cubic feet per second.	Annual rainfall in feet.	Discharges in cubic feet per second.	Annual rainfall in feet.	Discharges in cubic feet per second.
·1	·0883	2·1	1·8550	4·1	3·6217
·2	·1766	2·2	1·9433	4·2	3·7100
·3	·2650	2·3	2·0317	4·3	3·7983
·4	·3533	2·4	2·1200	4·4	3·8866
·5	·4417	2·5	2·2083	4·5	3·9750
·6	·5300	2·6	2·2966	4·6	4·0633
·7	·6183	2·7	2·3850	4·7	4·1517
·8	·7066	2·8	2·4733	4·8	4·2400
·9	·7950	2·9	2·5617	4·9	4·3283
1·0	·8833	3·0	2·6500	5·0	4·4166
1·1	·9717	3·1	2·7383	5·5	4·8583
1·2	1·0600	3·2	2·8266	6·	5·3000
1·3	1·1483	3·3	2·9150	6·5	5·7417
1·4	1·2366	3·4	3·0033	7·	6·1833
1·5	1·3250	3·5	3·0917	7·5	6·6250
1·6	1·4133	3·6	3·1800	8·	7·0666
1·7	1·5017	3·7	3·2683	8·5	7·5083
1·8	1·5900	3·8	3·3566	9·	7·9500
1·9	1·6783	3·9	3·4450	9·5	8·3917
2·0	1·7666	4·0	3·5333	10·	8·8333

TABLE II.—PART 3.

Discharges in cubic feet per second, equivalent to a given daily rainf
(24 hours) over catchment areas.

Area in sq. miles.	For a daily rainfall in feet and decimals of								
	·1	·09	·08	·07	·06	·05	·04	·03	·02
	Cubic feet per second.								
1	32·26	29·03	25·81	22·58	19·34	16·13	12·90	9·67	6·45
2	64·52	58·07	51·62	45·16	38·72	32·26	25·81	19·36	12·90
3	92·80	83·52	74·24	64·96	55·68	48·40	37·12	27·84	18·56
4	129·0	116·1	103·2	90·30	76·40	64·50	51·60	38·70	25·80
5	161·3	145·2	129·0	112·9	96·50	80·64	64·50	48·40	32·25
6	193·5	174·2	154·8	135·4	116·1	96·78	77·40	58·06	38·70
7	225·8	203·2	160·6	158·0	135·5	112·9	90·30	67·73	45·15
8	258·0	232·2	206·4	180·6	154·8	129·0	103·2	77·40	51·60
9	290·4	261·4	232·3	203·3	174·3	145·2	116·2	87·13	58·10
10	322·6	290·3	258·3	235·8	193·5	161·3	129·2	96·77	64·60

Area in square miles.	For a daily rainfall in feet and decimals of								
	·0833	·075	·0666	·0583	·05	·0417	·033	·025	·016
	or its equivalent in inches and decimals of								
	1·0	·9	·8	·7	·6	·5	·4	·3	·2
	Cubic feet per second.								
1	26·89	24·20	21·51	18·82	16·13	13·44	10·75	8·07	5·38
2	53·78	48·40	43·00	37·64	32·26	26·89	21·50	16·13	10·75
3	80·67	54·60	64·53	56·47	45·40	40·33	32·26	24·20	16·13
4	107·5	96·75	86·00	75·25	64·50	53·78	43·00	32·25	21·50
5	134·4	120·9	107·5	94·08	80·64	67·22	53·75	40·32	26·87
6	161·3	145·1	135·0	112·9	96·78	80·67	67·55	48·39	33·77
7	188·2	169·3	150·5	131·7	112·9	94·11	75·25	56·45	37·62
8	215·1	193·6	172·1	150·5	129·0	107·5	86·05	64·50	43·02
9	242·0	217·8	193·6	169·4	145·2	121·0	96·80	72·60	48·40
10	268·9	242·0	215·1	187·4	161·3	134·4	107·5	80·65	53·75

[redacted]

TABLE III.—Part 1.

Guide for capacity of reservoirs and supply from gathering grounds.

upply required, aring 240 days r eight months.	Contents of reservoir to hold that supply.	Surface of that reservoir if 3 feet deep on the average.	Catchment area necessary to fill that reservoir in four months, having one foot available rainfall in that time.
Cub. ft. per second.	Cubic feet.	Square feet.	Square miles.
1	20 736 000	6 912 000	·7438
2	41 472 000	13 824 000	1·4876
3	62 208 000	20 736 000	2·2314
4	82 944 000	27 648 000	2·9752
5	103 680 000	34 560 000	3·7190
6	124 416 000	41 472 000	4·4628
7	145 152 000	48 384 000	5·2066
8	165 888 000	55 296 000	5·9504
9	186 624 000	62 208 000	6·6942
10	207 360 000	69 120 000	7·4380
1·3444	27 878 400	9 292 800	1
2·6888	55 756 800	18 585 600	2
4·0333	83 635 200	27 878 400	3
5·3777	111 513 600	37 171 200	4
6·7222	139 392 000	46 464 000	5
8·0666	167 270 400	55 756 800	6
9·4100	195 148 800	65 049 600	7
10·7555	223 027 200	74 342 000	8
12·0999	250 905 600	83 635 200	9
13·4444	278 784 000	92 928 000	10

N.B.—The next page will contain two examples for this table.

EXAMPLE I.

A discharge of 18 234 cubic feet per second is wanted during eigh months of the year from a reservoir which is to be supplied by catchment area yielding an available rainfall of 1·32 feet during th remaining four months ; required the contents of the reservoir, an the size of the catchment area.

Obtain from the Table the quantities due to 1 foot of rainfall,

Supply, cubic feet per second.	Contents of reservoir, cubic feet.	Catchment area, square miles.
10	207 360 000	7·4380
8	165 888 000	5·9504
·2	4 147 200	·1487
·03	622 080	·0223
·004	82 944	·0029
18·234	378 100 224	3·5623

$$\text{Catchment area for } 1\cdot32 \text{ feet of fall} = \frac{13\cdot5623}{1\cdot32} = 10\cdot274 \text{ sq. miles.}$$

EXAMPLE II.

A catchment area of 21·963 square miles, having an available rainfall of 1·32 feet in four months of rainy season, supplies a reservoir which is to hold water for eight months' supply ; what should be the full contents of the reservoir, and the supply in cubic feet per second during the eight months ?

The proportionate catchment area for an available rainfall of one foot will = 21·963 × 1·32 = 29·001 square miles.

Area	Contents of reservoir, cubic feet.	Supply, cub. ft. per second.
20	557 568 000	26·888
9	250 905 600	12·0999
·001	27 878	·0013
29·001	808 501 478	38·9892

[redacted]

TABLE III.—Part 2.

Guide for acreage under irrigation, and for population under water-supply.

Cub. feet per second.	At 50 acres per cub. foot per second.	At 75 acres per cub. foot per second.	At 100 acres per cub. ft. per second.	At 150 acres per cub. ft. per second.	At 200 acres per cub. ft. per second.	At 250 acres per cub. ft. per second.	At 300 acres per cub. ft. per second.
	Number of acres watered.						
1	50	75	100	150	200	250	300
2	100	150	200	300	400	500	600
3	150	225	300	450	600	750	900
4	200	300	400	600	800	1000	1200
5	250	375	500	750	1000	1250	1500
6	300	450	600	900	1200	1500	1800
7	350	525	700	1050	1400	1750	2100
8	400	600	800	1200	1600	2000	2400
9	450	675	900	1350	1800	2250	2700
10	500	750	1000	1500	2000	2500	3000

Cub. feet per second.	At 5 gallons per head daily.	At 7½ gallons per head daily.	At 10 gallons per head daily.	At 15 gallons per head daily.	At 20 gallons per head daily.	At 25 gallons per head daily.	At 30 gallons per head daily.
	Population supplied.						
1	107732	71820	53866	35910	26933	21546	17955
2	215464	143640	107732	71820	53866	43093	35910
3	323196	215460	161598	107730	80799	64639	53865
4	430928	287280	215464	143640	107732	86186	71820
5	538660	359100	269330	179550	134665	107932	89775
6	646392	430920	323196	215460	161598	129278	107730
7	754124	474740	377062	237370	188531	150825	118685
8	861856	574560	430928	287280	215464	172371	143640
9	969588	646380	484794	323190	242397	193917	161595
10	1077820	718200	538660	359100	269330	215464	179550

N.B.—The next page will contain explanatory examples.

Example I.

A combined irrigation and water-work scheme yields 18·234 cubic feet per second; what amount of land and of population could it supply, at the rates of 150 acres per cubic foot per second, and 7½ gallons per head per diem, if one-fourth alone is to be used for the water-works?

The supply available for irrigation will be = 18·234 − 4·558 = 13·676 cubic feet per second; and from Table III., Part 2, we obtain the required results, thus—

Cubic feet per second.	Population.	Cubic feet per second.	Acres.
4.	287 280	10·	1500
·5	35 910	3·	450
·05	0 591	·6	90
·008	574	·07	10·5
		·006	·9
4·558	327 355	13·676	2051·

Example II.

A town has a population of 40,000, requiring water supply at 15 gallons per head daily, and has suburbs to the extent of 1,400 acres requiring irrigation at 150 acres per cubic foot per second of supply:—what catchment area will be necessary to provide this, if the annual rainfall is 60 inches?

According to Part 2, Table III., the supply necessary will be

For population.		For irrigation.		Total cubic feet per second.
35 910	1	1 350·	9·	
3 591	·1	50	·04	
489	·02			
40 000	1·12	1 400	9·04	10·16

Now, assuming that out of 60 inches annual rainfall, 30 can be utilized after deducting for all losses:—we find that according to Part 2, Table II., this is equivalent to a supply of 2·2083 cubic feet per second from one square mile, hence the minimum catchment area necessary will $= \frac{10\cdot16}{2\cdot208} = 4\cdot6$ square miles.

[redacted]

TABLE IV.—PART 1.

Table of flood discharges in cubic feet per second, due to catchment areas in square miles, and corresponding to a coefficient $n=1$ in the formula

$$Q = n \times 100 (N)^{\frac{3}{4}}.$$

For local values of coefficients, see Part 2, Table XII.

Catchment area.	Flood discharge.	Catchment area.	Flood discharge.	Catchment area.	Flood discharge.	Catchment area.	Flood discharge.
·01	3	11	604	41	1620	71	2446
·02	5	12	645	42	1650	72	2472
·03	7	13	685	43	1679	73	2498
·04	9	14	724	44	1708	74	2523
·05	11	15	762	45	1737	75	2549
·06	12	16	800	46	1766	76	2574
·07	14	17	837	47	1795	77	2599
·08	15	18	874	48	1824	78	2625
·09	16	19	910	49	1852	79	2650
		20	946	50	1880	80	2675
·1	18	21	981	51	1908	81	2700
·2	30	22	1016	52	1936	82	2725
·3	41	23	1050	53	1964	83	2750
·4	50	24	1084	54	1992	84	2775
·5	59	25	1118	55	2020	85	2799
·6	68	26	1151	56	2047	86	2824
·7	76	27	1184	57	2074	87	2849
·8	85	28	1217	58	2802	88	2873
·9	92	29	1250	59	2129	89	2898
		30	1282	60	2155	90	2922
1·	100	31	1314	61	2183	91	2946
2·	168	32	1345	62	2210	92	2971
3·	238	33	1377	63	2236	93	2995
4·	283	34	1408	64	2263	94	3019
5·	334	35	1439	65	2289	95	3043
6·	383	36	1470	66	2316	96	3067
7·	430	37	1500	67	2342	97	3091
8·	476	38	1531	68	2368	98	3115
9·	520	39	1561	69	2394	99	3139
10·	562	40	1590	70	2420	100	3162

TABLE IV.—Part 1—*continued.*

Catchment area.	Flood discharge.	Catchment area.	Flood discharge.	Catchment area.	Flood discharge.	Catchment area.	Flood discharge.
110	3307	410	9112	710	13 754	1250	21 062
120	3625	420	9278	720	13 900	1500	24 108
130	3850	430	[illegible]	730	14 044	1750	27 057
140	4070	440	9607	740	14 188	2000	29 907
150	4286	450	9770	750	14 332	2500	35 355
160	4499	460	9933	760	14 475	3000	40 536
170	4709	470	10 094	770	14 617	3500	45 504
180	4914	480	10 255	780	14 760	4000	50 297
190	5117	490	10 415	790	14 901	4500	54 943
200	5318	500	10 574	800	15 042	5000	59 460
210	5517	510	10 732	810	15 183	5500	63 867
220	5712	520	10 890	820	15 324	6000	68 173
230	5906	530	11 046	830	15 463	6500	72 391
240	6098	540	11 202	840	15 603	7000	76 529
250	6287	550	11 357	850	15 742	7500	[illegible]
260	6475	560	11 512	860	15 881	8000	84 590
270	6661	570	11 666	870	16 019	8500	88 525
280	6845	580	11 819	880	16 157	9000	92 402
290	7027	590	11 791	890	16 295	9500	96 448
300	7208	600	12 123	900	16 432	10 000	100 000
310	7388	610	12 204	910	16 568		
320	7566	620	12 425	920	16 705	20 000	168 179
330	7743	630	12 575	930	16 841	30 000	238 285
340	7918	640	12 724	940	16 976	40 000	282 855
350	8092	650	12 873	950	17 112	50 000	334 370
360	8265	660	13 021	960	17 246	60 000	383 366
370	8436	670	13 169	970	17 381	70 000	430 352
380	8607	680	13 316	980	17 511	80 000	475 683
390	8776	690	13 463	990	17 649	90 000	519 615
400	8944	700	13 609	1000	17 783	100 000	562 341

[redacted]

TABLE IV.—Part 2.

d discharges from catchment areas with a coefficient $n = 8·25$ and corresponding waterway for bridge openings.

(By Colonel Dickens.)

tchment area.	Flood discharge, co-eff 8·25	Assumed velocity.	Flood waterway.	No. of sq. openings.	Span.	Height of pier.
quare miles.	Cub. feet per sec.	Feet.	Square feet.	No.	Feet.	Feet.
·0016	6·5	5	1·5	1	1½	1
·0031	11·	5	2·25	1	2	1⅛
·0047	15	5	3·	1	2	1½
·0078	22	5	4·5	1	3	1½
·0125	31	5	6·	1	3	2
·0250	52	5	10·5	1	4	2¾
·0625	103	6	18·	1	6	3
·1250	173	6	29·	1	7	4
·2500	292	6	49·	1	10	5
·5000	490	6	81·	1	12	7
1	825	7	137	2	12	6
2	1 388	7	200	3	12	6
3	1 881	7	270	3	14	7
5	2 760	7	400	3	16	8
7	3 550	7	507	3	18	9
10	4 640	7	663	3	20	11
20	7 804	8	975	5	20	10
30	10 577	8	1 322	5	24	11
50	15 605	9	1 734	5	30	11½
100	26 094	9	2 899	5	40	14½
200	43 884	10	4 388	7	40	15½
300	59 481	10	5 948	9	40	16½
500	87 255	10	8 725	9	50	19
000	146 737	10	14 673	15	50	19
000	246 780	11	22 434	15	60	24
000	334 487	11	30 408	20	60	25
000	490 636	12	40 886	20	75	27
000	825 000	12	68 750	30	75	30
000	1 387 746	13	106 749	40	75	35
000	1 870 962	13	143 920	45	80	40
000	2 695 690	14	190 256	50	90	42
000	4 639 274	15	309 285	60	100	50

TABLE V.

Comparative, usual, and safe bottom velocities.

	Feet per second.		Feet per second.
Slow rivers	·33	Sailing ships	17
Ordinary rivers	2·25	Sea steamers	20
Rapid rivers	10·25	River steamers	26
A man's walk	4·5	Railways, English	47
Horse trot	10·25	„ French	40
Racing speed	88	„ American	27
Winds	10·25	Sound at 30°	1090
Storms	52·75	Sound at 63°	1122
Hurricanes	117·25	Air into a vacuum. Bar. 30″	1344

	Feet per second.
Limits usual for canals	1 to 4
Limits for rivers and canals just navigable	3 to 4½
Limits for irrigating channels	1 to 3
Limits for sewers and brick conduits	1 to 4½
Earthenware drainage pipes	4½
Maximum tidal current measured	15
Best velocities for pipes, so as to get a maximum discharge under pressure	25 to 35

Safe maximum bottom velocities.	Feet per second.
For soft clay	·25
For fine sand	·5
For coarse sand and small gravel	·7
For gravel as large as beans	1·
For gravel one inch in diameter	2·25
For pebbles one and a half inches in diameter	3·33
For heavy shingle	4·
For softer rocks, brick and earthenware	4·5
For hard rock	6 to 10

TABLE VI.—PART 1.

Ordinary limits of channel gradients.

Reciprocal of slope.	
1 in 500 000	Least canal slope to produce motion.
1 in 16 000 1 in 6 000	Limits of tidal navigation for large canals.
1 in 15 000 1 in 5 000	Fall of most deltaic or inundation canals.
1 in 6 000 1 in 2 000	Fall of most canals.
1 in 3 000 1 in 1 000	Fall of smaller canals, channels.
1 in 5 000 1 in 500	Fall of most rivers.
1 in 300 1 in 80	Fall of torrents.

Maximum gradients.

1 in 50	Ordinary railways.
1 in 30	Turnpike road.
1 in 20	Public road.
1 in 16	Private road.
1 in 8	Maximum for an ordinary carriage to ascend.
1 in 4	Maximum for beasts of burden.
1 in 1½	Maximum for hill walking.

Various slopes.

1 to 1 to ½ to 1	Chalk; dry clay.
1 to 1	Compact earth rubble, dry set.
1¼ to 1	Gravel, shingle, dry sand.
1½ to 1	Average mixed earth, dry.
1¾ to 1	Vegetable earth, dry.
2 to 1	Sand dry.
2 to 1	Minimum for slated and tiled roofs.
2½ to 1	Maximum for back slopes of rammed earthen dams.
3½ to 1	Maximum for breast slopes of rammed earthen dams.
4 to 1 to 3 to 1	Wet clay, peat.

N.B.—Wetted soil requires a less slope than dry soil generally.

TABLE VI.—Part 2.

Reduction of gradients.

Slope (S)	Fall of one in	Fall in feet per mile.	Slope (S)	Fall of one in	Fall in feet per mile.
·000 0100	100 000	·0528	·000 55	1818	2·904
·000 0133	75 000	·0704	·000 6	1666	3·168
·000 0150	66 666	·0792	·000 65	1538	3·332
·000 0200	50 000	·1056	·000 66	1500	3.52
·000 0250	40 000	·1820	·000 7	1429	3·696
·000 0300	33 333	·1584	·000 75	1333	3·960
·000 0333	30 000	·1760	·000 8	1250	4·224
·000 0350	28 571	·1848	·000 85	1176	4·488
·000 0400	25 000	·2112	·000 9	1111	4·752
·000 0450	22 222	·2376	·000 95	1053	5·016
·000 0473	21 120	·2500			
·000 0500	20 000	·2640	·001	1000	5·28
·000 0600	16 666	·3168	·00110	909	5·808
·000 0700	14 286	·3696	·00111	900	5·864
·000 0800	12 500	·4224	·00125	800	6·6
·000 0900	11 111	·4752	·00143	700	7·54
·000 0947	10 560	·5	·0015	666	7·92
·000 1000	10 000	·528	·00166	600	8·8
·000 1111	9000	·5866	·00175	571	9·24
·000 1250	8000	·6600	·002	500	10·56
·000 1420	7004	·7500			
·000 1428	7000	·7543			
·000 1500	6666	·7920	·00225	444	11·88
·000 1666	6000	·8800	·0025	400	13·20
·000 1750	5714	·9240	·00275	364	14·52
·000 1894	5280	1·	·003	333	15·84
·000 2000	5000	1·056	·00325	308	16·66
			·00333	300	17·60
·000 25	4000	1·320	·0035	286	18·48
·000 3	3333	1·584	·00375	266	19·80
·000 333	3000	1·760	·004	250	21·12
·000 35	2857	1·848	·00425	235	22·44
·000 4	2500	2·112	·0045	222	23·76
·000 45	2222	2·376	·00475	210	25·08
·000 5	2000	2·640	·005	200	26 40

[redacted]

TABLE VI.—Part 2—*continued.*

Reduction of gradients.

	Fall of one in	Fall in feet per mile.	Slope S.	Fall of one in	Fall in feet per mile.
	200	26·40	·015385	65·	81·23
63	190	27·78	·0155	64·5	81·84
	181·8	29·04	·016	62·5	84·48
55	180	29·33	·0165	60·6	87·12
32	170	31·05	·016667	60·	88·
	166·66	31·68	·017	58·8	89·76
50	160	33·	·0175	57·1	92·40
	153·8	33·32	·018	55·6	95·04
67	150	35·20	·018182	55·	96·
	142·86	36·96	·0185	54·1	97·68
43	140	37·71	·019	52·6	100·32
	133·3	39·60	·0195	51·3	102·96
92	130	40·60	·02	50·	105·6
	125	42·25			
33	120	44·	·021	47.6	110·88
	117·6	44·88	·022	45·4	116·16
	111·1	47·52	·023	43·5	121·44
91	110	48·	·024	41·7	126·72
	105·3	50·16	·025	40·	132·
	100	52·80	·026	38·5	137·28
			·027	37·0	142·56
	95·2	55·44	·028	35·7	147·84
26	95	55·57	·029	34·5	153·12
	90·9	58·08	·03	33·3	158·4
11	90	58·66			
	86·9	60·72	·031	32·3	163·68
55	85	62·11	·032	31·3	168·96
	83·3	63·36	·033	30·3	174·24
	80	66·	·034	29·4	179·52
	76·9	68·64	·035	28·5	184·8
3	75	70·40	·036	27·8	190·08
	74·1	71·28	·037	27·0	195·36
	71·4	73·92	·038	26·3	200·64
36	70	75·42	·039	25·6	205·92
	66·7	79·20	·04	25·	211·2

TABLE VI.—Part 3.

Reduction table for angular slopes.

Angular Slope.	Ratio to one perpendicular.	Reduction in feet and decimals for 100 feet.	Angular Slope.	Ratio to one perpendicular.	Reductio feet an decimals 100 fe
1°	57	·015	5° 30′	...	·46
1° 15′	[illegible]	...	5° 42′	10	...
1° 30′	39	·034	5° 45′	...	·50
1° 45′	33	...	6°	9·5	·54
2°	28	·061	6° 15′	...	·59
2° 15′	25	...	6° 21′	9	...
2° 30′	23	·095	6° 30′	...	·64
2° 45′	21	...	6° 43′	8·5	...
3°	19	·137	6° 45′	...	·699
3° 15′	18	·161	7°	...	·745
3° 28′	17	...	7° 7′	8	...
3° 30′	...	·187	7° 15′	...	·800
3° 35′	16	...	7° 30′	...	·856
3° 45′	...	·214	7° 36′	7·5	...
3° 49′	15	...	7° 45′	...	·913
4°	...	·244	8°	...	·973
4° 6′	14	...	8° 8′	7	...
4° 15′	...	·275	8° 15′	...	1·035
4° 24′	13	...	8° 30′	...	1·098
4° 30′	...	·308	8° 45′	6·5	1·164
4° 45′	12	·343	9°	...	1·231
5°	11·5	·381	9° 15′	...	1·300
5° 12′	11	...	9° 27′	6	...
5° 15′	...	·420	9° 30′	...	1·371
5° 27′	10·5	...	9° 45′	...	1·444

[redacted]

TABLE VI.—Part 3—*continued.*

Reduction table for angular slopes.

gular ope.	Ratio to one perpendicular.	Reduction in feet and decimals for 100 feet.	Angular Slope.	Ratio to one perpendicular.	Reduction in feet and decimals for 100 feet.
52′	5·75	...	17° 6′	3·25	...
°	...	1·519	17° 30′	...	4·628
° 18′	5·5	...	18°	...	4·894
° 30′	...	1·675	18° 26′	3	...
° 47′	5·25	...	18° 30′	...	5·168
°	...	1·837	19°	...	5·448
° 19′	5	...	19° 30′	...	5·736
° 30′	...	2·008	19° 59′	2·75	...
° 53′	4·75	...	20°	...	6·031
2°	...	2·185	21° 48′	2·5	...
2° 30′	...	2·370	23° 58′	2·25	...
2° 32′	4·5	...	25°	...	9·369
3°	...	2·553	26° 34′	2	...
3° 15′	4·25	...	29° 44′	1·75	...
3° 30′	...	2·763	30°	...	13·397
4°	4	2·970	33° 41′	1·5	...
4° 2′	...	...	35°	...	18·085
4° 30′	...	3·185	38° 39′	1·25	...
4° 55′	3·75	...	40°	...	23·396
5°	...	3·407	45°	1	...
5° 30′	...	3·637	50°	...	35·721
5° 56′	3·5	...	53° 8′	·75	...
6°	...	3·874	56° 20′	·66	...
6° 30′	...	4·118	60°	...	50·
7°	...	4·370	63° 26′	·5	...

in feet.	other values.	·001	·0005	·000 33	·000 25	·0002	·000 166	·000 143	·000 125	·000 111	·0001
		Velocities of discharge in feet per second.									
·05	22·3607	·707	·5	·409	·353	·316	·289	·267	·25	·236	·224
·1	31·6228	1·	·707	·577	·5	·447	·408	·378	·353	·833	·316
·15	38 7298	1·225	·866	·707	·612	·547	·5	·463	·433	·408	·387
·20	44·7214	1·414	1·	·816	·707	·632	·577	·534	·5	·471	·447
·25	50·	1·581	1·118	·913	·790	·707	·645	·597	·559	·527	·5
·3	54·7723	1·732	1·225	·999	·866	·775	·706	·655	·612	·577	·548
·35	59·1608	1·871	1·323	1·081	·935	·837	·764	·707	·661	·624	·592
·4	63·2456	2·	1·414	1·154	1·	·894	·816	·756	·707	·666	·632
·45	67·0820	2·121	1·500	1·224	1·060	·949	·865	·802	·750	·707	·671
·5	70·7107	2·236	1·581	1·290	1·118	1·	·912	·845	·790	·745	·707
·55	74·1620	2·345	1·658	1·354	1·172	1·049	·957	·886	·829	·782	·742
·6	77·4597	2·449	1·732	1·431	1·224	1·095	1·	·926	·866	·816	·775
·65	80·6226	2·550	1·803	1·472	1·275	1·140	1·041	·964	·901	·850	·806
·7	83·6663	2·646	1·871	1·528	1·323	1·183	1·080	1·	·935	·882	·837
·75	86·6025	2·739	1·936	1·581	1·369	1·225	1·118	1·035	·968	·913	·866
·8	89·4427	2·828	2·	1·633	1·414	1·265	1·155	1·069	1·	·943	·894
·85	92·1955	2·915	2·062	1·683	1·457	1·304	1·190	1·101	1·031	·972	·922
·9	94·8683	3·	2·121	1·732	1.5	1·342	1·225	1·132	1 060	1·	·949
·95	97·4679	3·082	2·179	1·779	1·541	1·378	1·257	1·164	1·089	1·027	·975
1·00	100·	3·162	2·236	1·815	1·581	1·414	1·283	1·195	1·118	1·054	1·

TABLE VII.—*continued.*

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·000166	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
1·1	104·8809	3·317	2·345	1·915	1·658	1·483	1·354	1·254	1·172	1·106	1·049
1·2	109·5445	3·464	2·449	2·	1·732	1·549	1·414	1·310	1·224	1·155	1·095
1·3	114·0175	3·606	2·550	2·082	1·803	1·612	1·472	1·363	1·275	1·202	1·140
1·4	118·3216	3·742	2·646	2·160	1·871	1·673	1·527	1·414	1·323	1·247	1·183
1·5	122·4745	3·873	2·739	2·236	1·936	1·732	1·581	1·464	1·369	1·291	1·225
1·6	126·4911	4·	2·828	2·309	2·	1·789	1·633	1·511	1·414	1·333	1·265
1·7	130·3840	4·123	2·915	2·380	2·061	1·844	1·683	1·558	1·457	1·374	1·304
1·8	134·1641	4·243	3·	2·449	2·121	1·897	1·732	1·604	1·5	1·414	1·342
1·9	137·8405	4·359	3·082	2·517	2·179	1·949	1·779	1·648	1·541	1·453	1·378
2·	141·4214	4·472	3·162	2·582	2·236	2·	1·825	1·691	1·581	1·491	1·414
2·1	144·9138	4·583	3·240	2·646	2·291	2·049	1·871	1·732	1·620	1·528	1·449
2·2	148·3240	4·690	3·317	2·707	2·345	2·098	1·914	1·773	1·658	1·563	1·483
2·3	151·6575	4·796	3·391	2·769	2·398	2·145	1·958	1·812	1·695	1·599	1·517
2·4	154·9193	4·899	3·464	2·828	2·449	2·191	1·999	1·852	1·732	1·633	1·549
2·5	158·1139	5·	3·536	2·886	2·5	2·236	2·040	1·889	1·768	1·666	1·581
2·6	161·2452	5·099	3·606	2·943	2·549	2·280	2·081	1·927	1·803	1·699	1·612
2·7	164·3168	5·196	3·674	3·	2·598	2·324	2·121	1·964	1·837	1·732	1·643
2·8	167·3320	5·292	3·742	3·055	2·646	2·366	2·160	2·	1·871	1·764	1·673
2·9	170·2939	5·385	3·808	3·109	2·692	2·408	2·198	2·035			

Mean radius in feet.	multiplied by √S for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·00016	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
3·1	176·0682	5·568	3·937	3·215	2·784	2·490	2·273	2·105	1·968	1·856	1·761
3·2	178·8854	5·657	4	3·266	2·828	2·530	2·309	2·138	2·	1·886	1·789
3·3	181·6590	5·745	4·062	3·317	2·872	2·569	2·345	2·172	2·031	1·915	1·817
3·4	184·3909	5·831	4·123	3·367	2·915	2·608	2·382	2·204	2·061	1·944	1·844
3·5	187·0829	5·916	4·183	3·416	2·958	2·646	2·415	2·236	2·092	1·972	1·871
3·6	189·7367	6·	4·243	3·464	3·	2·683	2·449	2·267	2·121	2·	1·897
3·7	192·3538	6·083	4·301	3·512	3·041	2·720	2·483	2·299	2·150	2·028	1·924
3·8	194·9359	6·164	4·359	3·559	3·082	2·757	2·516	2·330	2·179	2·055	1·949
3·9	197·4842	6·245	4·416	3·606	3·122	2·793	2·548	2·360	2·208	2·082	1·975
4·	200	6·325	4·472	3·651	3·162	2·828	2·581	2·390	2·236	2·108	2·
4·1	202·4846	6·403	4·528	3·696	3·202	2·864	2·613	2·421	2·264	2·134	2·025
4·2	204·9390	6·481	4·583	3·741	3·240	2·898	2·645	2·450	2·291	2·160	2·049
4·3	207·3644	6·557	4·637	3·786	3·278	2·933	2·680	2·480	2·318	2·186	2·074
4·4	209·7618	6·633	4·690	3·829	3·316	2·966	2·707	2·507	2·345	2·211	2·098
4·5	212·1320	6·708	4·743	3·873	3·354	3·	2·738	2·535	2·371	2·236	2·121
4·6	214·4761	6·782	4·796	3·916	3·391	3·033	2·769	2·564	2·398	2·261	2·145
4·7	216·7948	6·856	4·848	3·958	3·428	3·066	2·798	2·591	2·424	2·285	2·168
4·8	219·0890	6·928	4·899	4·	3·464	3·098	2·828	2·619	2·449	2·309	2·191
4·9	221·3594	7·	4·950	4·041	3·5	3·130	2·857	7·646	2·475	2·333	2·214
5·	223·6068	7·071	5·	4·082	3·535	3·162	2·886	2·672	2·5	2·357	2·236

TABLE VII.—*continued.*

Hydraulic mean radius in feet	Tabular No. to be multiplied by √S for other values	Values of S.									
		·001	·0005	·00033	·00025	·0002	·000166	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
5·1	225·8318	7·141	5·050	4·122	3·570	3·194	2·914	2·699	2.525	2·380	2·258
5·2	228·0351	7·211	5·099	4·164	3·605	3·225	2·944	2·725	2·549	2·404	2·280
5·3	230·2173	7·280	5·148	4·204	3·640	3·258	2·972	2·751	2·574	2·427	2·302
5·4	232·3790	7·348	5·196	4·242	3·674	3·286	2·999	2·777	2·598	2·440	2·324
5·5	234·5208	7·416	5·244	4·282	3·708	3·317	3·027	2·803	2·622	2·472	2·345
5·6	236·6432	7·483	5·292	4·320	3·742	3·347	3·054	2·828	2·646	2·494	2·366
5·7	238·7467	7·550	5·339	4·359	3·775	3·376	3·080	2·854	2·669	2·517	2·387
5·8	240·8319	7·616	5·385	4·397	3·808	3·406	3·109	2·878	2·692	2·539	2·408
5·9	242·8992	7·681	5·431	4·434	3·840	2·435	3·135	2·903	2·715	2·560	2·429
6·	244·9490	7·746	5·477	4·472	3·873	3·464	3·162	2·928	2·738	2·582	2·449
6·1	246·9818	7·810	5·523	4·508	3·905	3·493	3·187	2·952	2·761	2·603	2·470
6·2	248·9980	7·874	5·568	4·546	3·937	3·521	3·214	2·977	2·784	2·625	2·490
6·3	250·9980	7·937	5·612	4·583	3·968	3·550	3·240	3·	2·806	2·646	2·510
6·4	252·9822	8·	5·657	4·619	4·	3·578	3·264	3·024	2·828	2·666	2·530
6·5	254·9510	8·062	5·701	4·654	4·031	3·606	3·290	3·048	2·850	2·687	2·550
6·6	256·9047	8·124	5·745	4·690	4·062	3·633	3·316	3·071	2·872	2·708	2·570
6·7	258·8436	8·185	5·788	4·725	4·093	3·661	3·340	3·093	2·894	2·728	2·588
6·8	260·7681	8·246	5·831	4·761	4·123	3·688	3·366	3·117	2·915	2·749	2·608
6·9	262·6785	8·307	5·874	4·796	4·153	3·715	3·391	3·138	2·937	2·769	2·627
7·	264·5751	8·367	5·916	4·830	4·184	3·742	3·415	3·162	2·957	2·789	2·646

mean radius in feet.	multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·000166	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
7·1	266·4583	8·426	5·958	4·865	4·213	3·768	3·439	3·185	2·979	2·809	2·665
7·2	268·3282	8·485	6·	4·898	4·242	3·795	3·463	3·207	3·	2·828	2·683
7·3	270·1851	8·544	6·042	4·933	4·272	3·821	3·483	3·229	3·021	2·848	2·702
7·4	272·0294	8·602	6·083	4·966	4·301	3·847	3·511	3·252	3·041	2·867	2·720
7·5	273·8618	8·660	6·124	5·	4·330	3·873	3·535	3·273	3·062	2·887	2·739
7·6	275·6810	8·718	6·164	5·033	4·359	3·899	3·558	3·296	3·082	2·906	2·757
7·7	277·4887	8·775	6·205	5·066	4 387	3·924	3·582	3·317	3·102	2·925	2·775
7·8	279·2848	8·832	6·245	5·099	4·416	3·950	3·605	3·339	3·122	2·944	2·793
7·9	281·0694	8·888	6·285	5·132	4·444	3·975	3·628	3·360	3·142	2·963	2·811
8·	282·8427	8·944	6·325	5·163	4·472	4·	3·650	3·380	3·162	2·981	2·828
8·1	284·6050	9·	6·364	5·196	4·5	4·025	3·674	3·400	3·182	3·	2·846
8·2	286·3564	9·055	6·403	5·229	4·527	4·050	3·697	3·423	3·201	3·018	2·863
8·3	288·0972	9·110	6·442	5·261	4·555	4·074	3·719	3·443	3·221	3·037	2·881
8.4	289·8275	9·165	6·481	5·291	4·582	4·099	3·741	3·464	3·240	3·055	2·898
8·5	291·5476	9·220	6·519	5·322	4·610	4·123	3·763	3·485	3·259	3·073	2·915
8·6	293·2576	9·274	6·557	5·354	4·637	4·147	3·785	3·505	3·278	3·091	2·933
8·7	294·9576	9·327	6·595	5·385	4·663	4·171	3·807	3·525	3·297	3·109	2·950
8·8	296·6479	9·381	6·633	5·416	4·690	4·195	3·829	3·545	3·316	3·127	2·966
8·9	298·3287	9·434	6·671	5·447	4·717	4·219	3·851	3·566	3·335	3·145	2·983
9·	300·	9·487	6·708	5·477	4·743	4·243	3·872	3·586	3·354	3·162	3·

TABLE VII.—*continued.*

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·000166	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
9·1	301·6621	9·539	6·745	5·506	4·769	4·266	3·893	3·606	3·372	3·179	3·017
9·2	303·3150	9·592	6·782	5·537	4·796	4·290	3.915	3·625	3·391	3·197	3·033
9·3	304·9590	9·644	6·819	5·568	4·822	4·313	3.936	3·645	3·409	3·215	3·050
9·4	306·5942	9·695	6·856	5·599	4·847	4·336	3.958	3·665	3·428	3·232	3·066
9·5	308·2207	9·747	6·892	5·630	4·873	4·359	3.980	3·685	3·446	3·249	3·082
9·6	309·8387	9·798	6·928	5·658	4·899	4·382	4·	3·704	3·464	3·266	3·098
9·7	311·4482	9·849	6·964	5·686	4·924	4·405	4·020	3 723	3·482	3·283	3·114
9·8	313·0495	9·899	7·	5·714	4·949	4·427	4·039	3·741	3·5	3·299	3·130
9·9	314·6427	9·950	7·036	5·745	4·975	4·450	4·060	3·761	3·518	3·317	3·146
10	316·2278	10·	7·071	5·773	5·	4·472	4·082	3·778	3·535	3·333	3·162
15	387·2983	12·247	8·660	7·070	6·123	5·477	4·998	4·629	4·330	4·082	3·873
20.	447·2136	14·142	10·	8·165	7·071	6·325	5·773	5·346	5·	4·714	4·472
25	500·	15·811	11·180	9·128	7·905	7·071	6·453	5·977	5·590	5·270	5·
30	547·7226	17·321	12·247	10·	8·660	7·746	7·070	6·546	6·123	5 773	5·477
35	591·6080	18·708	13·229	10·801	9·354	8·367	7·636	7·071	6·614	6·236	5·916
40	632·4555	20·	14·142	11·545	10·	8·944	8·162	7·559	7·071	6·666	6·325
45	670·8204	21·213	15·	12·247	10·606	9·487	8·659	8·017	7·5	7·071	6·708
50	707·1068	22·361	15·811	12·910	11·180	10.	9·127	8·456	7·905	7·454	7·071
60	774·5967	24·495	17·321	14·142	12·242	10·954	10·	9·258	8·660	8·165	7·746
70	836·6600	26·458	18·708	15·275	13·229	11·832	10·799	10·	9·354	8·819	8·367

EXPLANATORY EXAMPLES TO TABLE VII.

EXAMPLE 1. A river has a hydraulic radius of 5·2 feet, a hydraulic ope of ·0002 and a cross section of 1000 square feet, required the charge, assuming a frictional co-efficient of ·03.

By Table VII. the unmodified mean velocity of discharge = 3·225 t per second, and by Part 3 of Table XII. the value of c the -efficient suitable to this radius and slope is ·668, hence the true discharge = $c \times A \times Q = \cdot 66 \times 1000 \times 3\cdot 225 = 2128$ cubic feet per cond.

EXAMPLE 2. Suppose the river mentioned in the last example to ve a hydraulic slope of ·0015, the remaining data being as before, quired the discharge.

In this case the inclination not being one of those given at the heads columns, make use of the tabular number corresponding to the ydraulic radius, which is 228·03, and multiplying it by $\sqrt{\cdot 0015}$, an nmodified mean velocity of discharge 8·87 feet per second is obtained. aking the suitable co-efficient c from Part 3, Table XII., the true scharge = $c \times A \times V = \cdot 65 \times 1000 \times 8\cdot 87 = 5765$ cubic feet r second.

EXAMPLE 3. A canal is to have a cross section of 250 square feet, a ydraulic radius of 4 feet, and must discharge when in perfect order d regimen 500 cubic feet per second, what is the hydraulic slope ecessary, and what will its discharge be when it wears itself into a ate resembling a natural channel, if we assume the other data to remain the same?

From an inspection of Table VII. and the table of co-efficients for tificial channels, it appears that for the given radius a co-efficient of 753 and a slope of ·00018 would nearly satisfy the conditions: assung ·753, the mean velocity becomes 2·65 and the slope ·000175. The scharge for a natural channel would require the co-efficient ·632, and ould $= \cdot 632 \times 250 \times 200 \times \sqrt{\cdot 000175} = 418$ cubic feet per econd.

TABLE VIII.

For full cylindrical tubes—Pipes, Sewers, &c.

PART 1.—Discharges in cubic feet per second.

$$Q = c \times 39{\cdot}27 \left(Sd^5\right)^{\frac{1}{2}}$$

PART 2.—Diameters in feet and decimals.

$$d = \frac{1}{c^{\frac{2}{5}}} \times {\cdot}23 \left(\frac{Q^2}{S}\right)^{\frac{1}{5}}$$

PART 3.—Heads for a length of 100 feet, in feet.

$$H = \frac{1}{c^2} \times {\cdot}0648 \, \frac{Q^2}{d^5}$$

being values of the corresponding formulæ, when $c = 1$.

N.B.—For more correct results, apply the values of the co-efficie (c) given in Part 3, Table XII., in every case, using the table of usef numbers, Part 7, Table XII., for powers and roots.

The tabular numbers extend the use of the tables to any slope.

Some explanatory examples follow this table.

[redacted]

TABLE VIII.—Part 1.

Discharges through full cylindrical tubes, Pipes, Culverts, &c.

liameters n feet.	For slopes of one in 100	150	200	300	400	500	1000	Tabular No. to be multiplied by $\sqrt{S}$ for other slopes.
	Discharges in cubic feet per second.							
1″) ·083	·008	·006	·006	·005	·004	·004	·003	·079
2″) ·166	·04	·04	·03	·03	·02	·02	·01	·445
3″) ·25	·12	·10	·09	·07	·06	·05	·04	1·227
4″) ·33	·25	·21	·18	·15	·13	·11	·08	2·519
5″) ·416	·44	·36	·31	·25	·22	·20	·14	4·401
6″) ·5	·69	·57	·49	·40	·35	·31	·22	6·939
7″) ·583	1·02	·83	·72	·59	·51	·46	·32	10·206
8″) ·66	1·43	1·16	1·01	·82	·71	·64	·45	14·251
9″) ·75	1·91	1·56	1·35	1·10	·97	·86	·61	19·128
0″) ·83	2·49	2·03	1·76	1·44	1·25	1·11	·79	24·895
1″) ·916	3·16	2·58	2·23	1·82	1·58	1·41	1·00	31·594
2″) 1·00	3·93	3·28	2·78	2·27	1·96	1·76	1·24	39·27
1·25	6·86	5·60	4·85	3.96	3·43	3·07	2·16	68·601
1·5	10·82	8·82	7·65	6·25	5·41	4·84	3·42	108·216
1·75	15·91	12·99	11·25	9·18	7·95	7·11	5·03	159·095
2·	22·21	18·14	15·71	12·83	11·11	9 93	7·02	222·146
2·25	29·82	24·35	21·08	17·22	14·91	13·34	9·43	298·505
2·5	38·81	31·69	27·44	22·41	19·40	17·35	12·27	388·078
2·75	49·25	40·22	34·82	28·43	24·62	22 02	15·57	492·489
3·	61·21	49·99	43·28	35·31	30·61	27·37	19·35	612·105
3·25	74·77	61·04	52·87	43·18	37·38	33·44	23·64	747·744
3·5	89·99	73·49	63·63	51·96	44·99	40·25	28·46	899·990
3·75	106·94	87·33	75·61	61·74	53·46	47·82	33·81	1069·397
4·	125·66	102·63	88·84	72·55	62·83	56·20	39·73	1256·640
4·25	146·23	119·42	103·38	84·32	73·11	65·39	46·24	1462·262
4·5	168·69	137·76	119·26	97·39	84·34	75·44	53·34	1686·886
4·75	193·10	157·70	136·52	111·48	96·55	86·36	61·06	1931·028
5·	219·54	179·26	155·24	126·75	109·77	99·18	69·43	2195·436
5·5	278·61	227·48	197·00	160·85	139·30	124·60	88·10	2786·060
6·	346·31	282·76	244·88	199 94	173·16	154·88	109·51	3463·130
6·5	423·03	345·40	299·13	244·23	211·51	189·18	133·77	4230·262
7·	509·13	415·70	360·01	293·95	254·57	227·69	161·00	5091·322

TABLE VIII.—Part 2.

Diameters of full Pipes of small discharge and high inclination.

Discharges in cubic feet per second.	For slopes of one in 100	150	200	300	400	500	1000	Tabular No. to be multiplied by $(\frac{1}{s})^{\frac{1}{5}}$ for other slopes.
	Diameters in feet.							
·1	·23	·25	·26	·29	·30	·32	·36	·0916
·2	·30	·33	·35	·38	·40	·42	·48	·1208
·3	·36	·39	·41	·44	·47	·49	·57	·1421
·4	·40	·43	·46	·50	·53	·55	·63	·1594
·5	·44	·47	·50	·55	·58	·60	·69	·1743
·6	·47	·51	·54	·59	·62	·65	·75	·1875
·7	·50	·54	·58	·62	·66	·69	·79	·1994
·8	·53	·57	·61	·66	·70	·73	·84	·2104
·9	·56	·60	·64	·69	·73	·77	·88	·2215
1·	·58	·63	·66	·72	·76	·80	·92	·2300
1·1	·60	·65	·69	·75	·79	·83	·95	·2385
1·2	·62	·67	·71	·77	·82	·86	·99	·2474
1·3	·64	·70	·74	·80	·85	·89	1·02	·2556
1·4	·66	·72	·76	·82	·87	·91	1·05	·2631
1·5	·68	·74	·78	·85	·90	·94	1·08	·2705
1·6	·70	·76	·80	·87	·92	·96	1·11	·2776
1·7	·71	·77	·82	·89	·94	·99	1·13	·2844
1·8	·73	·79	·84	·91	·96	1·01	1·16	·2910
1·9	·75	·81	·86	·93	·99	1·03	1·18	·2973
2·0	·76	·83	·88	·95	1·01	1·05	1·21	·3035
2·1	·78	·84	·89	·97	1·03	1·07	1·23	·3095
2·2	·79	·86	·91	·99	1·04	1·09	1·26	·3153
2·3	·81	·87	·93	1·01	1·06	1·11	1·28	·3209
2·4	·82	·89	·94	1·02	1·08	1·13	1·30	·3265
2·5	·83	·90	·96	1·04	1·10	1·15	1·32	·3318
2·6	·85	·92	·97	1·05	1·12	1·17	1·34	·3368
2·7	·86	·93	·99	1·07	1·13	1·19	1·36	·3422
2·8	·87	·95	1·00	1·09	1·15	1·20	1·38	·3472
2·9	·88	·96	1·02	1·10	1·17	1·22	1·40	·3521
3·0	·90	·97	1·03	1·12	1·18	1·24	1·42	·3569

For special cases modify the discharge by a co-efficient (*c*) before applying it to the table, to find the diameter.

TABLE VIII.—Part 2—*continued.*

Diameters of full cylindrical Sewers, Drains of large discharge and low inclination.

charges bic feet second	For slopes of one in							Tabular No. to be multiplied by $\left(\frac{1}{s}\right)^{\frac{1}{5}}$ for other slopes.
	500	1000	1500	2000	2500	3000	4000	
	Diameters in feet.							
1	·80	·92	·99	1·05	1·10	1·14	1·21	·23
2	1·05	1·21	1·31	1·39	1·45	1·51	1·59	·30348
3	1·24	1 42	1·54	1·63	1·71	1·77	1·88	·35692
4	1·39	1·59	1·73	1 83	1 91	1·99	2·10	·40045
5	1·52	1·74	1·89	2·00	2·09	2·17	2 30	·43780
6	1·63	1·87	2 03	2·15	2·25	2·34	2 47	·47096
7	1·74	1·99	2·16	2·29	2·40	2·48	2·63	·50092
8	1·83	2·10	2 28	2 42	2·53	2·62	2·78	·52840
9	1·92	2·21	2·39	2·53	2·65	2·75	2·91	·55389
10	2 00	2.30	2·49	2·64	2·76	2 87	3·04	·57773
11	2·08	2·39	2·59	2·74	2·87	2·98	3·15	·60018
12	2·15	2·47	2·68	2·84	2·97	3·08	3·27	·62144
13	2 22	2·55	2·76	2·93	3·07	3 18	3·37	·64166
14	2·29	2·63	2 85	3·02	3·16	3·28	3·47	·66096
15	2·36	2·71	2·93	3·11	3·25	3·37	3·57	·67946
16	2·42	2·78	3·01	3·19	3·32	3·46	3·66	·69723
17	2·48	2·84	3·08	3·27	3·42	3·54	3·75	·71434
18	2·53	2.91	3·16	3·34	3·50	3 63	3·84	·73086
19	2·59	2·97	3 22	3·42	3·57	3·70	3 92	·74684
20	2·64	3·03	3·29	3·49	3·65	3·78	4·00	·76232
30	3·11	3·57	3·87	4·10	4·29	4·45	4·71	·89655
40	3·49	4·00	4·34	4·60	4·81	4·99	5·28	1·0059
50	3·81	4·38	4·75	5·03	5·26	5·45	5·78	1·0998
60	4·10	4·71	5·11	5·41	5·66	5·87	6·21	1·1830
70	4·36	5·01	5·43	5·75	6·02	6·24	6·61	1·2582
80	4·60	5·28	5·73	6·07	6·35	6·58	6 97	1·3273
90	4·82	5·54	6·00	6·36	6·65	6·90	7·31	1·3913
100	5·03	5·78	6·26	6·64	6·94	7·20	7·62	1·4512
200	6·64	7·62	8 27	8·76	9·16	9·50	10·06	1·9149
300	7·81	8·97	9·72	10·30	10·77	11·16	11·83	2 2520

For special cases, modify the discharge by a co-efficient (*c*) before plying it to the table, to find the diameter.

[redacted]

TABLE VIII.—Part 3.

Small Pipes. Heads for a length of 100 feet.

For discharges in cubic feet per second.	For diameters in feet. ·083 (1″)	·166 (2″)	·25 (3″)	·333 (4″)	·416 (5″)	Tabular number to be divided by d for other diameters.
	Head of water in feet.					
·1	161	5.04	·664	·157	·0516	·000648
·2	645	20·16	2·654	·630	·2064	·002592
·3	1451	45·35	5·972	1·417	·4644	·005832
·4	2580	80·62	10·617	2·525	·8256	·010368
·5	4031	125·97	16·589	3·937	1·2899	·0162
·6	5804	180·56	23·888	5·669	1·8575	·023328
·7	7900	246·90	32·514	7·716	2·5283	·031752
·8	10318	322·48	42·467	10·078	3·3023	·041472
·9	13061	408 15	53·748	12·754	4·1794	·052488
1·0	16124	503 89	66·355	15·746	5·1598	·0648
1·1	19510	609·70	80·290	19·053	6·2433	·078408
1·2	23219	725·59	95·551	22·675	7·4301	·093312
1·3	27250	851·56	112·140	26·611	8·7200	·109512
1·4	31604	987·60	130·056	30·863	10·1132	·127008
1·5	36280	1133·74	149·299	35·429	11·6095	·1458
1·6	41278	1289·94	169·869	40 311	13·2090	·165888
1·7	46599	1456·22	191·767	45·507	14·9118	·187272
1·8	52243	1632·59	214·992	51·018	16·7177	·209952
1·9	58209	1819 02	239·542	56·844	18·6268	·233928
2·0	64497	2015·54	265 421	62·986	20·6391	·2592
2·1	71108	2222·13	292·627	69·442	22·7546	·285768
2·2	78042	2438 80	321·160	76 212	24·9733	·313632
2·3	85298	2665·55	351·020	83·299	27·2952	·342792
2·4	92876	2902·36	382·206	90·699	29·7203	·373248
2·5	100777	3149·28	414·720	98·415	32·2487	·4050
2·6	109000	3406·26	448·561	106·446	34·8801	·438048
2·7	117546	3673·32	483·730	114·791	37·6147	·472392
2·8	126415	3950·46	520·225	123·452	40·4527	·508032
2·9	135605	4237·67	558·048	132·427	43 3937	·544968
3·0	145119	4534·96	597·197	141·717	46·4380	·5832

For special cases modify the discharge by a co-efficient (*c*) before applying it, to find the head necessary.

[redacted]

TABLE VIII.—Part 3—*continued.*

Pipes. Head for a length of 100 feet.

r dis- ges in ic feet second.	For diameters in feet. ·5 (6″)	·583 (7″)	·666 (8″)	·75 (9″)	·833 (10″)	Tabular number to be divided by d^5 for other diameters.
	Head of water in feet and decimals.					
·1	·0207	·0096	·0049	·0027	·0016	·000648
·2	·0829	·0384	·0197	·0107	·0064	·002592
·3	·1866	·0863	·0443	·0246	·0145	·005832
·4	·3318	·1535	·0787	·0437	·0258	·010368
·5	·5184	·2398	·1230	·0683	·0403	·0162
·6	·7465	·3454	·1772	·0989	·0580	·023328
·7	1·0163	·4701	·2411	·1338	·0790	·031752
·8	1·3271	·6140	·3149	·1748	·1032	·041472
·9	1·6796	·7753	·3995	·2212	·1306	·052488
1·0	2·0736	·9594	·4921	·2731	·1612	·0648
1·1	2·5091	1·1608	·5954	·3304	·1951	·078408
1·2	2·9860	1·3815	·7086	·3932	·2322	·093312
1·3	3·5044	1·6213	·8316	·4615	·2725	·109512
1·4	4·0643	1·8804	·9645	·5352	·3160	·127008
1·5	4·6656	2·1586	1·1072	·6144	·3628	·1458
1·6	5·3084	2·4560	1·2597	·6991	·4128	·165888
1·7	5·9927	2·7726	1·4221	·7892	·4660	·187272
1·8	6·7185	3·1084	1·5943	·8847	·5224	·209952
1·9	7·4857	3·4633	1·7764	·9858	·5821	·233928
2·0	8·2944	3·8375	1·9683	1·0674	·6450	·2592
2·1	9·1446	4·2309	2·1701	1·2042	·7111	·285768
2·2	10·0362	4·5377	2·3816	1·3216	·7804	·313632
2·3	10·9693	5·0751	2·6031	1·4445	·8530	·342792
2·4	11·9439	5·5260	2·8343	1·5729	·9288	·373248
2·5	12·9600	5·9961	3·0755	1·7067	1·0078	·4050
2·6	14·0175	6·4854	3·3264	1·8459	1·0900	·438048
2·7	15·1165	6·9939	3·5872	1·9906	1·1755	·472392
2·8	16·2570	7·5215	3·8579	2·1408	1·2642	·508032
2·9	17·4390	8·0683	4·1383	2·2965	1·3561	·544968
3·0	18·6624	8·6344	4·4287	2·4576	1·4512	·5832

For special cases modify the discharge by a co-efficient (*c*) before pplying it, to find the head necessary.

TABLE VIII.—Part 3—*continued.*

ylindrical Sewers or Tunnels. Head for a length of 100 feet.

dis- in feet cond.	For diameters in feet. 3	4	5	6	7	Tabular numbers to be divided by d^5 for other diameters.
	Head of water in feet.					
1	·0003	·00006	·00002	·000008	·000004	·06
2	·0011	·00025	·00008	·000033	·000015	·26
3	·0024	·00057	.00018	·000075	·000035	·58
4	·0043	·00101	·00033	·000133	·000062	1·04
5	·0067	·00158	·00052	·000208	·000096	1·62
6	·0096	·00228	·00074	·000300	·000139	2·33
7	·0131	·00310	·00102	·000408	·000189	3·18
8	·0167	·00405	·00133	·000533	·000247	4·15
9	·0216	·00513	·00168	·000675	·000312	5·25
10	·0267	·00633	.00207	·000833	·000386	6·48
15	·0600	·01424	·00466	·001875	·000868	14·58
20	·1067	·02531	·00829	·008333	·001542	25·92
25	·1667	·03955	·01296	·005208	·002410	40·50
30	·2400	·05695	·01866	·007500	·003470	58·32
35	·3267	·07752	·02540	·010208	·004723	79·38
40	·4267	·10132	·03318	·013333	·006169	103·68
45	·5400	·12815	·04199	·016875	·007875	131 22
[illegible]	·6667	·15823	·05184	·020833	·009639	162·00
55	·8067	·19143	·06273	·025208	·011663	196·02
60	·9600	·22781	·07465	·030000	·013880	233 28
65	1·1267	·26736	·08761	·035208	·016289	273.78
70	1·3067	·31008	·10161	·030833	·018892	317·52
75	1·5000	·35596	·11664	·046875	·021687	364·50
80	1 6678	·40500	·13271	·053333	·024675	414·72
85	1·9267	·45721	·14982	·060208	·027856	468·18
90	2·1600	·51258	·16796	·067500	·031230	524·88
95	2·4067	·57112	·18714	·075208	·034796	584·82
100	2·6667	·63281	·20736	·083333	·038555	648·
200	10·6667	2·53120	1·52944	·333333	·154222	2592·
300	24·	5·69530	1·86624	.750000	·346998	5832·

special cases, modify the discharge by a co-efficient (c) before ing it, to find *the head necessary*.

EXPLANATORY EXAMPLES TO TABLE VIII.

EXAMPLE 1. What is the discharge of an enamelled 3-inch pipe having a hydraulic slope of 1 in 400; and what would be its least full discharge when old, irrespectively of sectional obstruction?

By Table VIII., Part 1, the tabular discharge is ·06 cubic feet per second; and by the Table of co-efficients (Table XII., Part 3), for very smooth surfaces, including smooth plaster, and enamelled or glazed pipes, the co-efficient c for a pipe having this slope and a hydraulic radius, which for cylindrical pipes running full is one-fourth of the diameter, is ·84, the discharge when new is $= \cdot 84 \times \cdot 06 = \cdot 05$ cubic feet per second.

If preferred in any other unit, refer to Table XI., Part 2, by inspecting which we find it = 18 gallons per minute.

When the pipe is old its surface will not be rougher than that of ordinary metal, and taking the co-efficient for metal with this slope and radius to be ·61, the least discharge is $= \cdot 61 \times \cdot 06 = \cdot 037$ cubic feet per second, or 14 gallons per minute.

EXAMPLE 2. A masonry culvert has a diameter of 42 inches, and a slope of 1 in 200, what is its discharge when running full?

By Part 1, Table VIII., the tabular discharge is 63·63 cubic feet per second, and the co-efficient for this slope and a hydraulic radius of ·875 feet will according to Table XII. be 1·10; hence the actual discharge will be $1 \cdot 10 \times 63 \cdot 63 = 70$ cubic feet per second.

EXAMPLE 3. What must be the diameter of a cast iron pipe to discharge 20 cubic feet per second with a slope of one in 500?

By Part 2, Table VIII., the tabular diameter will be 2·64 feet and the hydraulic radius ·66 feet; turning to the table of co-efficients (Table XII., Part 3), we take $c = 1 \cdot 03$: and assuming a modified discharge $\frac{Q}{c} = 19 \cdot 4$, and referring again to Part 2, Table VIII., we obtain a true diameter = 2·62 feet.

EXAMPLE 4. What should be the dimensions of an ovoidal brickwork sewer to discharge 50 cubic feet per second with a slope of 1 in 1,000, the sewer flowing two-thirds full?

The co-efficient to modify the discharge through cylindrical into

[redacted]

t for ovoidal sewers of the usual type running two-thirds full is erally assumed to be $= \frac{35}{39\cdot27} = \cdot89$; hence the first modification discharge will be $\cdot89 \times 50 = 44\cdot5$: using this and referring to rt 2, Table VIII., we get a first approximation to a diameter of 4·19 t. Secondly, referring to the table of co-efficients, Table XII., we tain a co-efficient *c*, corresponding to a hydraulic radius of 1·05 and lope of ·001, = 1·13; and modifying the discharge a second time becomes $= \frac{44\cdot5}{1\cdot13} = 39\cdot4$ giving, according to Part 2, Table VIII., 7 feet for the diameter of a cylindrical sewer. Hence the dimen- ns for the corresponding ovoidal sewer will be

d diameter of top circle $= 3\cdot97$

$\frac{d}{2}$ diameter of bottom circle $= 1\cdot98$

$\frac{3d}{2}$ radius of each side circle $= 5\cdot96$

$\frac{3d}{2}$ depth of sewer $= 5\cdot96$

Example 5. A series of enamelled pipes has a total head of 30 feet, d consists of 3600 feet of 8-inch pipe, 2100 feet of 6-inch, and 600 t of 5-inch; required the discharge and head necessary for each pipe.

Assume any discharge as 1 cubic foot per second, and obtaining the parate tabular heads due to it, divide the total head in the same oportion.

$$\begin{array}{rl@{\qquad}rl}
\cdot4921 \times 36 = & 17\cdot72 & 17\cdot72 \times 30 \div 92 = & 5\cdot77 \\
2\cdot0736 \times 21 = & 43\cdot55 & 43\cdot55 \times 30 \div 92 = & 14\cdot15 \\
5\cdot1598 \times 6 = & 30\cdot95 & 30\cdot95 \times 30 \div 92 = & 10\cdot08 \\
\text{Total} = & 92\cdot22 & \text{Total} = & 30\cdot
\end{array}$$

nd modifying these by the squares of the suitable co-efficients, obtain tual heads for a first approximation.

$$\begin{array}{rl@{\qquad}rl}
5\cdot77 \div (\cdot95)^2 = & 6\cdot41 & 6\cdot41 \times 30 \div 39\cdot22 = & 4\cdot90 \\
14\cdot15 \div (\cdot87)^2 = & 18\cdot62 & 18\cdot62 \times 30 \div 39\cdot22 = & 14\cdot24 \\
10\cdot08 \div (\cdot84)^2 = & 14\cdot19 & 14\cdot19 \times 30 \div 39\cdot22 = & 10\cdot86 \\
\text{Total} = & 39\cdot22 & \text{Total} = & 30\cdot
\end{array}$$

d the discharge $= \frac{1 \times \sqrt{30}}{\sqrt{92}} = \cdot57$ cubic feet per second $= 213$ gal- ns per minute.

EXAMPLE 6. A discharge of 300 gallons per minute is requir through a series of ordinary iron pipes composed of 800 yards 7-inch, 300 yards of 6-inch, and 100 yards of 5-inch; what is head required for each pipe?

By Tables of equivalents (Part 2, Table XI.), 300 gals. per min = ·8 cubic feet per second, and the corresponding tabular heads Part 3, Table VIII., can be taken as a first approximation, a modifying these by the squares of the suitable co-efficients given Table XII. we get the true heads thus:—

	Length. Head.	True Heads.
7 inch	$\cdot 6140 \times 24 = 14\cdot 74$	$14\cdot 74 \div (\cdot 66)^2 = 33\cdot 50$
6 inch	$1\cdot 3271 \times 9 = 11\cdot 94$	$11\cdot 94 \div (\cdot 63)^2 = 29\cdot 85$
5 inch	$3\cdot 3023 \times 3 = 9\cdot 91$	$9\cdot 91 \div (\cdot 61)^2 = 26\cdot 78$
	36·59 feet.	Total 90·13 fe

TABLE IX.

Gives velocities of discharge in feet per second for sluices, and fices, due to various heads for certain co-efficients, also theoretical locities to which any co-efficient may be applied; being an application of the formula

$$V = m \times 8{\cdot}025 \sqrt{H}.$$

ere for orifices H = depth of centre of motion of orifice.

The same table also applies to overfalls, weirs, and notches, but in is case using the same general formula, H is the depth from still ater to sill-level, and the velocities given in the table must be reduced one-third to obtain velocities of discharge for all sorts of overfalls.

For values of (m) the co-efficient, see Parts 5 and 6, Table XII.

This table can also be used for the converse purpose.

TABLE IX.

Head in feet.	CO-EFFICIENTS.					
	For natural velocity. ·1·	For narrow bridge-openings. ·9	For velocity of approach. ·8·	For special weirs. ·7·	For special orifices. ·6·	For lo crested dams. ·5·
	Velocities of Discharge.					
·01	·803	·722	·642	·562	·482	·401
·02	1·135	1·021	·908	·794	·681	·567
·03	1·390	1·251	1·112	·973	·834	·695
·04	1·605	1·445	1·284	1·123	·963	·803
·05	1·794	1·615	1·435	1·256	1·076	·897
·06	1·966	1·769	1·573	1·376	1·180	·983
·07	2·123	1·911	1·698	1·486	1·274	1·062
·08	2·270	2·043	1·816	1·589	1·362	1·135
·09	2·408	2·167	1·926	1·686	1·445	1·204
·1	2·538	2·284	2·030	1·777	1·523	1·269
·2	3·589	3·230	2·871	2·512	2·153	1·794
·3	4·395	3·956	3·516	3·078	2·637	2·198
·4	5·075	4·568	4·060	3·553	3·045	2·537
·5	5·675	5·108	4·540	3·973	3·405	2·837
·6	6.216	5·594	4·973	4·351	3·730	3·108
·7	6·714	6·043	5·371	4·700	4·028	3·352
·8	7·178	6·460	5·742	5·025	4·307	3·589
·9	7·613	6·852	6·090	5·329	4·568	3·807
1	8·025	7·223	6·420	5·618	4·815	4·013

N.B.—For overfalls, reduce the tabular velocity by one-third.

[redacted]

TABLE IX.

Head in feet.	CO-EFFICIENTS.					
	For wide bridge-openings. ·96	For lock sluices. ·84	For special weirs. ·727	For weirs generally. ·666	For orifices generally. ·62	For special weirs. ·55
	Velocities of Discharge.					
·01	·770	·674	·584	·535	·498	·441
·02	1·089	·953	·825	·756	·704	·624
·03	1·334	1·168	1·011	·926	·862	·765
·04	1·541	1·348	1·167	1·069	·995	·883
·05	1·722	1·507	1·304	1·185	1·112	·987
·06	1·887	1·651	1·429	1·309	1·219	1·081
·07	2·038	1·783	1·543	1·414	1·316	1·169
·08	2·179	1·907	1·650	1·512	1·407	1·249
·09	2·311	2·023	1·751	1·604	1·493	1·324
·1	2·436	2·132	1·845	1·690	1·574	1·396
·2	3·445	3·014	2·609	2·390	2·225	1·973
·3	4·219	3·694	3·195	2·927	2·725	2·418
·4	4·872	4·264	3·689	3·380	3·147	2·792
·5	5·448	4·768	4·126	3·780	3·519	3·121
·6	5·968	5·221	4·519	4·140	3·854	3·419
·7	6·445	5·640	4·881	4·471	4·163	3·687
·8	6·890	6·030	5·218	4·781	4·450	3·948
·9	7·308	6·395	5·535	5·070	4·720	4·187
1	7·704	6·742	5·834	5·345	4·976	4·414

N.B.—For overfalls, reduce the tabular velocity by one-third.

[redacted]

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS.					
	For natural velocity. 1	For narrow openings of bridges. ·9	For velocity of approach. ·8	For special weirs. ·7	For special orifices. ·6	Fo c
	Velocities of Discharge.					
1·	8·0250	7·223	6·420	5·618	4·815	4
1·25	8·9722	8·075	7·178	6·281	5·383	4
1·5	9·8286	8·846	7·863	6·880	5·897	4
1·75	10·6161	9·554	8·493	7·431	6·370	5
2·	11·3491	10·214	9·079	7·944	6·809	5
2·25	12·0375	10·834	9·630	8·426	7·223	6
2·5	12·6886	11·420	10·151	8·882	7·613	6
2·75	13·3079	11·977	10·646	9·316	7·985	6
3·	13·8997	12·510	11·120	9·730	8·340	6
3·25	14·4673	13·020	11·574	10·127	8·680	7
3·5	15·0134	13·512	12·010	10·509	9·008	7
3·75	15·5403	13·986	12·432	10·878	9·324	7
4·	16·0500	14·445	12·840	11·235	9·630	8
4·25	16·5439	14·890	13·235	11·581	9·926	8
4·5	17·0235	15·322	13·619	11·916	10·214	8
4·75	17·4901	15·741	13·992	12·243	10·494	8
5·	17·9444	16·150	14·355	12·561	10 767	8
5·25	18·3876	16·549	14·710	12·871	11·033	9
5·5	18·8203	16·938	15·056	13·174	11·292	9
5·75	19·2433	17·319	15·395	13·470	11·546	9
6·	19·6572	17·691	15·726	13 760	11·794	9
6·25	20·0625	18·057	16 050	14·044	12·038	10
6·5	20·4598	18·414	16·368	14·322	12·276	10
6·75	20·8496	18·765	16·680	14·595	12·510	10
7·	21·2322	19·109	16·986	14·863	12·739	10
7·25	21·6079	19·447	17·286	15·126	12·965	10
7·5	21·9774	19·779	17·582	15·384	13·186	10
7·75	22 3406	20·107	17·873	15·638	13·404	11
8·	22·6981	20·428	18·158	15·889	13·619	11

N.B.—For overfalls, reduce the tabular velocity by one-thi

TABLE IX.—*continued.*

ead in feet.	CO-EFFICIENTS.					
	For wide bridge-openings.	For lock sluices.	For special weirs.	For weirs generally.	For orifices generally.	For special orifices.
	·96	·84	·727	·666	·62	·55
	Velocities of Discharge.					
1	7·704	6·741	5·836	5·345	4·975	4·413
1·25	8·614	7·537	6·525	5·976	5·562	4·934
1·50	9·436	8 256	7·147	6·546	6·109	5·420
1·75	10·192	8·918	7·720	7·071	6·582	5·839
2	10·895	9·533	8·253	7·558	6·936	6·241
2·25	11·556	10·112	8·754	8·017	7·461	6·621
2·50	12·181	10·659	9·227	8·451	7·867	6·978
2·75	12·776	11 179	9·678	8·863	8 251	7·319
3	13·344	11·676	10·108	9·257	8·618	7·645
3·25	13 889	12·153	10 521	9·635	8·825	7·957
3·50	14·413	12·612	10·918	9·999	9·308	8·258
3·75	14·919	13·054	11·301	10·350	9·635	8·547
4	15·408	13·482	11 672	10·689	9·951	8·827
4·25	15·882	13 897	12·027	11·018	10·257	9·099
4·50	16·343	14·300	12·380	11·338	10·554	9·363
4 75	16 800	14 695	12·718	11 651	10·846	9·622
5	17·227	15·074	13·049	11·952	11·121	9·865
5 25	17·652	15·446	13·372	12·247	11·400	10·113
5·50	18·068	15·809	13·686	12·534	11·669	10·351
5·75	18·474	16·165	13·994	12·817	11·931	10 584
6	18·871	16·512	14·295	13·092	12·188	10·812
6 25	19·260	16·853	14·590	13 362	12 439	11·034
6 50	19 642	17·187	14·879	13·627	12 685	11·253
6·75	20·016	17·514	15·162	13·886	12·927	11·467
7	20·383	17·835	15·440	14·141	13·164	11·688
7·25	20·744	18·151	15·714	14·391	13 402	11·889
7·50	21·099	18·461	15·982	14·637	13·626	12·082
7·75	21 447	18·767	16·246	14·879	13·851	12·287
8	21·791	19 067	16·506	15·117	14·073	12·484

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS.					
	For natural velocity. 1	For narrow bridge-openings. ·9	For velocity of approach. ·8	For special weirs. ·7	For special orifices. ·6	For broad-crested dams. ·5
	Velocities of Discharge.					
8·25	23·051	20·746	18·441	16·135	13·831	11·526
8·50	23·397	21·057	18·717	16·377	14·038	11·698
8·75	23·739	21·365	18·992	16·617	14·243	11·869
9	24·076	21·668	19·261	16·853	14·445	12·038
9·25	24·408	21·966	19·526	17·085	14·645	12·204
9·50	24·735	22·261	19·788	17 316	14·841	12·367
9·75	25·050	22·553	20·047	17·541	15·035	12·529
10	25·378	22·840	20·302	17·764	15·227	12·689
10·5	26·005	23·404	20·804	18·203	15·603	13·002
11	26·617	23·955	21·293	18·631	15·970	13·308
11·5	27·215	24·493	21 772	19·050	16·329	13·607
12	27·800	25·020	22·240	19·460	16·680	13·900
12·5	28·373	25·535	22·698	19·861	17·024	14·186
13	28·935	26·041	23·148	20·254	17·361	14·467
13·5	29·486	26·545	23·596	20·646	17·697	14 747
14	30·027	27·024	24·021	21·019	18·016	15·013
14·5	30·559	27·503	24·447	21·391	18·335	15·279
15	31·081	27·973	24·864	21·756	18·648	15·540
15·5	31·594	28·434	25·275	22·115	18·956	15·797
16	32·101	28·891	25·681	22·470	19·261	16·050
16·5	32·598	29·338	26·078	22·818	19·555	16·299
17	33·089	29·780	26·471	23·162	19·853	16·544
17·5	33·572	30·214	26·857	23·500	20·143	16·786
18	34·048	30·643	27·238	23·833	20·429	17·024
18·5	34·515	31·066	27·614	24·162	20·711	17·259
19	34·981	31·483	27·985	24·486	20·988	17·490
19·5	35·438	31·894	28·350	24·806	21·263	17·719
20	35·889	32.300	29·711	25·122	21·533	17·944

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS.					
	For wide bridge-openings.	For lock sluices.	For special weirs.	For weirs generally.	For orifices generally.	For special weirs.
	·96	·84	·727	·666	·62	·55
	Velocities of Discharge.					
8·25	22·129	19·362	16·762	15·352	14·292	12·677
8·50	22·461	19·654	17·014	15·582	14·506	12·867
8·75	22·789	19·941	17·263	15·810	14·718	13·056
9	23·112	20·223	17·508	16·034	14·927	13·242
9·25	23·431	20·502	17·749	16·256	15·133	13·424
9·50	23·746	20·778	17·987	16·473	15·336	13·604
9·75	24·056	28·049	88·223	16·689	15·536	13·782
10	24·363	21·317	18·455	16·902	15·734	13·958
10·5	24·964	21·844	18·910	17·112	16·123	14·302
11	25·552	22·358	19·355	17·727	16·502	14·639
11·5	26·126	22·860	19·791	18·125	16·873	14·968
12	26·688	23·352	20·216	18·515	17·236	15·290
12·5	27·238	23·834	20·633	18·897	17·591	15·605
13	27·778	24·306	21·042	19·271	17·940	15·914
13·5	28·307	24·769	21·442	19·637	18·287	16·222
14	28·826	25·223	21·836	19·998	18·617	16·514
14·5	29·337	25·670	22·222	20·352	18·946	16·807
15	29·838	26·108	22·602	20·700	19·270	17·094
15·5	30·331	26·540	22·976	21·042	19·588	17·377
16	30·817	26·965	23·344	21·379	19·903	17·655
16·5	31·294	27·383	23·706	21·711	20·207	17·929
17	31·765	27·794	24·062	22·037	20·515	18·198
17·5	32·229	28·200	24·413	22·358	20·815	18·465
18	32·686	28·600	24·760	22·676	21·110	18·726
18·5	83·137	28·995	25·101	22·988	21·391	18·985
19	33·582	29·384	25·438	23·298	21·688	19·239
19·5	34·021	29·768	25·771	23·602	21·991	19·491
20	34·454	30·147	26·091	23·902	22·251	19·739

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS.					
	For natural velocity. 1	For narrow openings of bridges. ·9	For velocity of approach. ·8	For special weirs. ·7	For special orifices. ·6	For broad-crested dams. ·5
	Velocities of Discharge.					
20·5	36·936	32·702	29·068	25·485	21·601	18·468
21	36·776	33·098	29·420	25·743	22·066	18·388
21·5	37·211	33·490	29·768	26·047	22·327	18·605
22	37·641	33·877	30·112	26·348	22·585	18·820
22·5	38·067	34·260	30·453	26·646	22·840	19·033
23	38·487	34·647	30·797	26·943	23·098	19·243
23·5	38·903	35·012	31·122	27·232	23·342	19·451
24	39 315	35·383	31·452	27·520	23·589	19·657
24·5	39·723	35·750	31·778	27·806	23·834	19·861
25	40·126	36·113	32·100	28·088	24·075	20·063
25·5	40·525	36·472	32·420	28·367	24·315	20·262
26	40·921	36·379	32·737	28·644	24·553	20·460
26·5	41·312	37·180	33·049	28·918	24·787	20·656
27	41·700	37·530	33·360	29·190	25·020	20·850
27·5	42·084	37·875	33·667	29·458	25·250	21·042
28	42·465	38·218	33 972	29·725	25·479	21·232
28·5	42·843	38·558	34·275	29·990	25·706	21·421
29	43·216	38·890	34·573	30·248	25·927	21·606
29·5	43·586	39·229	34·870	30·511	26·153	21·794
30	43·956	39·560	35·164	30·779	26·374	21·978
30·5	44·320	39·888	35·456	31·024	26·592	22·160
31	44·682	40 213	35·745	31·277	26·809	22·340
31·5	45·041	40·537	36·032	31·528	27·025	22·520
32	45·397	40·857	36·317	31·778	27·238	22·698
32·5	45·751	41·176	36·601	32 025	27·451	22·875
33	46·101	41·491	36·880	32·270	27·660	23·050
33·5	46·449	41·804	37·159	32·514	27·869	23·224
34	46·794	42·114	37·435	32·755	28·076	23·397
34·5	47·137	42 423	37·709	32·996	28·282	23·568
35	47·478	42·730	37·982	33·234	28·487	23·739

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

CO-EFFICIENTS.					
For wide bridge-openings.	For lock sluices.	For special weirs.	For weirs generally.	For orifices generally.	For special weirs.
·96	·84	·727	·666	·62	·55
Velocities of Discharge.					
34·882	30·522	26·423	24·199	22·528	19·985
35·305	30·892	26·737	24·493	22·701	20·227
35·723	31·257	27·060	24·783	22·971	20·465
36·136	31·619	27·373	25·069	23·337	20·702
36·544	31·976	27·682	25·353	23·601	20·936
36·948	32·329	27·988	25·633	23·868	21·228
37·347	32·679	28·291	25·910	24·120	21·396
37·743	33·025	28·590	26·184	24·375	21·623
38·134	33.367	28·886	26·455	24·628	21·847
38·521	33·706	29·180	23·724	24·878	22·069
38·904	34·041	29·470	26·990	25·125	22·288
39·284	34·373	29·757	27·253	25·371	22·506
39·660	34·702	30·042	27·514	25·613	22·722
40·032	35·028	30·324	27·761	25·854	22·935
40·401	35·351	30·604	28·028	26·092	23·146
40·767	35·671	30·881	28·282	26·328	23·355
41·129	35·988	31·155	28·533	26·563	23·563
41·488	36·302	31·427	28·782	26·891	23·766
41·844	36·614	31·697	29·029	27·024	23·973
42·197	36·923	31·956	29·274	27·253	24·176
42·548	37·229	32·230	29·517	27·478	24·376
42·895	37·533	32·493	29·758	27·703	24·574
43·240	87·835	32·754	29·997	27·925	24·772
43·581	38·134	33·013	30·234	28·146	24·968
43·920	38·430	33·270	30·470	28·365	25·162
44·257	38·725	33·525	30·703	28·582	25·355
44·591	39·017	33·778	30·935	28·798	25·546
44·923	39·307	34·029	31·165	29·012	25·737
45·252	39·595	34·278	31·393	29·225	25·925
45·578	39·881	34·526	31·620	29·436	26·113

—For overfalls, reduce the tabular velocity by one-third.

Explanatory Examples to Table IX.

Example 1.

An orifice 6 inches in diameter, has its centre under a head of 5 feet of water; required its discharge.

For a circular orifice using ·62 for a co-efficient, the velocity of discharge is 11·121 feet per second, and the sectional area, according to Part 7, Table XII., being ·1963, the discharge = ·1963 × 11·121 = 2·1836 cubic feet per second.

Example 2.

A rectangular orifice is 8 inches broad and 4 inches deep, and is under a head of 4 feet 3 inches; required its discharge.

Since the breadth is greater than the depth, a special co-efficient is required (*See* Co-efficients in Table XII.).

$$\text{Here } \frac{H}{L} = \frac{4{\cdot}25}{{\cdot}66} = 7 \text{ approximately, and } \frac{D}{L} = \frac{{\cdot}33}{{\cdot}66} = {\cdot}5.$$

These require a co-efficient ·612, which must hence be applied to the tabular discharge for natural velocity due to the co-efficient 1·00 ∴ the discharge = 16·544 × ·22 × ·612 = 2·227 cubic feet per second.

Example 3.

The fall of water through a bridge, having a sectional area of 500 square feet, is ·05 feet; required the discharge.

Take ·96 as a co-efficient for a wide opening, and we get the discharge = 1·758 × 500 = 879 cubic feet per second.

Example 4.

The difference of level between the upper and lower ponds of a canal is 6 feet, and the communicating sluice is 2 feet square; required its discharge.

[redacted]

ing the co-efficient ·84 and height 6;
ischarge is 16·512 × 4 = 66·048 cubic feet per second.
e effective head gradually decreasing, the mean discharge due to
eight is 33·024 cubic feet per second.
the lock is 60 long and 20 broad, it will hold 7,200 cubic feet of
, and at the above rate will be filled in 218 seconds, or about
minutes and a half.

AMPLE 5. Required the diameter of a vertical pipe to discharge 2
feet per second from a reservoir under a head of 30 feet.
ing the co-efficient ·84, we obtain from the Table 36·923 as velo-
f discharge.

e section will then $= \frac{2\cdot}{36\cdot923} =$ 05417 square feet,

will require a diameter of 3¼, or practically, 4 inches.

AMPLE 6. Required the length of a weir to discharge 5696 cubic
er second, at a depth or head from still water to sill of 4 feet.
th a co-efficient ·666, the tabular velocity of discharge is 10·689,
which one-third has to be deducted to obtain the mean velocity
charge over a weir.

nce $V = 10\cdot689 - 3\cdot563 = 7\cdot126$ feet per second,

he section $= \frac{5696}{7\cdot126} =$ nearly 800 feet;

the length $= \frac{\text{section}}{\text{depth}} =$ nearly 200 feet.

AMPLE 7. A river passes over a drowned weir: the upper level
ter is 3 feet above the lower level, and is 4 feet above the sill of
eir, which is 100 feet long; required the discharge.
e upper portion may be considered as a simple overfall with a
$H = 3$, and with a co-efficient ·666: the lower portion as an
, with the same head, but a co-efficient ·62.
ording to the Table the velocity of discharge for the one is
— 3·086 = 6·171 feet per second; and that for the other is
feet per second. Hence the discharge:

$$= 50\,(6\cdot171 \times 3 + 8\cdot618 \times 1) = 50 \times 27\cdot131$$
$$= 1356 \text{ cubic feet per second.}$$

Example 8. It is required to raise the upper portion of a river feet by means of a drowned weir across it. The river has a discha of 812 cubic feet per second, and a width of 70 feet; what must be height of the dam—1st, neglecting velocity of approach; 2nd, tak it at 2·5 feet per second?

1st. Let d = depth of sill of dam below the lower water.

Then V = velocity of upper portion, or true overfall;
= $\frac{2}{3}$ velocity for head 1·5 to a co-efficient ·666;
= 4·364 feet per second (from Table);
and V¹ = velocity of lower portion of orifice;
= velocity for a head 1·5 to a co-efficient ·62;
= 6·109 feet per second (from Table).

Then the total discharge 812, is as in the last Example

$$= 70 \left\{ V \times 1{\cdot}5 + V^1 \times d \right\} = 70\,(6{\cdot}546 + d \times 6{\cdot}109)$$

$$\text{hence } d = \frac{5{\cdot}054}{6{\cdot}109} = {\cdot}827 \text{ feet.}$$

2nd. Taking into consideration the velocity of approach and mo fying the co-efficients (*vide* Table XII.) accordingly.

The head due to velocity of approach 2·5 feet per second, for a co- efficient ·8, is from Table IX. about ·15 feet.

Hence the modified co-efficient for overfall will be

$$m \left\{ \left\{ 1 + \frac{h^{\frac{3}{2}}}{H} \right\} - \left\{ \frac{h^{\frac{3}{2}}}{H} \right\} \right\} = {\cdot}666 \left\{ \left\{ 1 + \frac{{\cdot}15^{\frac{3}{2}}}{1{\cdot}5} \right\} - \left\{ \frac{{\cdot}15^{\frac{3}{2}}}{1{\cdot}5} \right\} \right\}$$

$$= {\cdot}666 \left\{ (1{\cdot})^{\frac{3}{2}} - ({\cdot}1)^{\frac{3}{2}} \right\} = {\cdot}745$$

and the modified co-efficient for orifice will be

$$m \sqrt{\left\{ 1 + \frac{{\cdot}15}{1{\cdot}5} \right\}} = m\sqrt{1{\cdot}1} = {\cdot}62 \times 1{\cdot}049 = {\cdot}648$$

Making use of these two co-efficients instead of ·666 and ·62 as in the first portion of the Example, we obtain other values.

$$V = 4{\cdot}894\text{; and } V^1 = 6{\cdot}385$$

$$\text{hence } \frac{812}{70} = 11{\cdot}6 = 1{\cdot}5\ V + d\,V^1 = 7{\cdot}341 + d \times 6{\cdot}385$$

$$\text{and } d = \frac{4{\cdot}259}{6{\cdot}385} = {\cdot}667 \text{ feet.}$$

BENDS AND OBSTRUCTIONS.

TABLE X.

Part I.—Giving loss of head in feet due to bends in pipes corresponding to certain discharges.—(Weisbach formula.)

Part II.—Giving loss of head due to bends in rivers corresponding to certain velocities.—(Mississippi formula.)

Part III.—Giving approximate rise of water in feet due to obstructions, bridges, weirs, &c.:—(the whole section of water being = 1), and corresponding to certain velocities. — (Dubuat formula.)

TABLE X.—Part 1.

Table giving loss of head due to one bend of 90° for pipes with different discharges. (Weisbach formula.)

Diameter of pipe.		Radius of bend.	Loss of Head of water in feet.												
			·01	·05	·1	·2	·3	·4	·5	·6	·7	·8	·9	1	2
			Corresponding to discharges in cubic feet per second of												
(1″)	·083	·5	·02	·04	·05	·06	·09	·10	·12	·13	·14	·15	·16	·17	·23
(2″)	·166	·5	·07	·15	·22	·30	·37	·43	·44	·53	·57	·61	·64	·68	·96
(3″)	·25	1·	·15	·34	·49	·69	·84	·97	1·08	1·19	1·28	1·37	1·46	1·53	2·17
(4″)	·33	1·	·26	·59	·84	1·18	1·45	1·66	1·87	2·05	2·21	2·36	2·51	2·64	3·74
(5″)	·416	1·5	·43	·96	1·33	1·93	2·36	2·66	3·05	3·34	3·61	3·86	4·00	4·31	6·09
(6″)	·5	1·5	·61	1·35	1·92	2·71	3·32	3·83	4·29	4·69	5·07	5·42	5·75	6·06	8·55
(7″)	·583	1·5	·81	1·81	2·56	3·62	4·34	5·12	5·73	6·28	6·78	7·25	7·69	8·10	11·46
(8″)	·66	1·5	1·06	2·38	3·34	4·76	5·83	6·69	7·52	8·24	8·90	9·52	10·03	10·64	15·05
(9″)	·75	1·5	1·32	2·94	4·16	5·89	7·21	8·33	9·31	10·20	11·02	11·78	12·49	13·17	18·58
(10″)	·83	1·5	1·57	3·52	4·98	7·04	8·63	9·96	11·18	12·20	13·18	14·09	14·94	15·75	22·27
(11″)	·916	1·5	1·89	4·22	5·96	8·43	10·33	11·92	13·34	14·61	15·78	16·87	17·89	18·86	26·67
(12″)	1·0	1·75	2·27	5·08	7·16	10·16	12·44	14·36	16·06	17·60	19·01	20·32	21·55	22·71	32·12
	1·25	2·	3·4	7·7	10·9	15·4	18·0	21·8	24·4	26·7	28·9	30·9	32·7	34·5	48·1
	1·5	2·5	5·0	11·3	15·9	22·6	27·6	31·9	35·7	39·1	42·2	45·1	47·9	50·4	71·4
	1·75	3·	6·9	15·5	21·9	30·9	37·9	43·7	48·9	53·5	57·8	61·8	65·6	69·1	97·7
	2·	4·	9·4	20·9	29·6	41·9	51·3	59·2	66·2	72·5	78·3	83·7	88·8	93·6	132·4
	2·5	5·	14·6	32·7	46·2	65·4	80·1	92·4	103·4	113·3	122·4	130·9	138·8	146·3	206·9
	3·0	6·	21·1	47·1	66·6	94·2	115·4	133·2	148·9	163·2	176·3	188·4	199·9	210·7	297·9
	3·5	7·	28·7	64·1	90·7	128·2	157·1	181·3	202·8	222·1	239·0	258·7	272·0	290·7	405·6
	4·	8·	37·3	83·9	118·7	167·9	205·6	237·4	265·4	290·8	314·1	335·8	356·1	375·4	530·9
	5·	10	58·5	130·9	185·1	261·7	320·5	370·1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[redacted]

TABLE X.—Parts 2 and 3.

Part 2.—Bends of Rivers. (Mississippi formula.)

For velocities in feet per second.	For deflections of 10° $\frac{n}{3}$	20° $\frac{n}{2}$	30° n	60° 2 n	90° 3 n	180° 6 n
	Loss of head in feet.					
1	·0006	·0009	·0019	·0037	·0096	·0112
2	·0025	·0037	·0075	·0149	·0234	·0448
3	·0056	·0084	·0168	·0336	·0503	·1007
4	·0099	·0149	·0298	·0597	·0895	·1790
5	·0155	·0233	·0466	·0933	·1399	·2798
6	·0224	·0335	·0671	·1343	·2014	·4028
7	·0305	·0457	·0914	·1827	·2741	·5483
8	·0398	·0596	·1194	·2387	·3581	·7162
9	·0503	·0755	·1511	·3021	·4532	·9064
10	0622	·0933	·1865	·3730	·5592	1·1190

Part 3.—Obstructions (Dubuat formula) when the hydraulic slope is less than ·001.

For velocities in feet per second.	For percentages of obstruction to whole channel section. ·1	·2	·3	·4	·5	·6
	Rise resulting in feet.					
1	·004	·009	·018	·031	·051	·089
2	·015	·034	·070	·120	·203	·355
3	·035	·085	·158	·270	·456	·798
4	·062	·150	·282	·480	·811	1·419
5	·097	·236	·439	·752	1·267	2·218
6	·140	·341	·634	1·080	1·824	3·193
7	·191	·463	·862	1·470	2·484	4·346
8	·249	·602	1·126	1·920	3·245	5·677
9	·315	·766	1·426	2·430	4·107	7·185
10	·389	·956	1·758	3·008	5·070	8·872

EXPLANATORY EXAMPLES TO TABLE X.

EXAMPLE 1. A series of pipes have to discharge 5 gallons p second; there are 7 bends in the portion that consists of 5-inc pipe, 4 in that of 6-inch pipe, and 8 in that of 7-inch pipe; what the total loss of head on account of these bends?

From Table XI. 5 gallons per second = ·8 cubic feet per secon and taking the heads separately from Table X.

feet.
7 bends in 5 inch give 7 × ·045 = ·315
4 „ „ 6 „ „ 4 × ·030 = ·120
8 „ „ 7 „ „ 8 × ·010 = ·080
Total loss of head = ·515

The head on the pipes must therefore not only be sufficient to forc ·8 cubic feet per second through the pipes under ordinary conditions but must also be increased by ·515 on account of bends.

EXAMPLE 2. A river has one bend of 20°, two of 30°, and one of 90°, what is the total loss of head expended in overcoming these bends when the velocity is 5 feet per second?

From Part 2, Table XII.

1 bend of 20° gives 1 × ·0233 = ·0233
2 „ „ 30° „ 2 × ·0466 = ·0932
1 „ „ 90° „ 1 × ·1399 = ·1399
Total head expended = ·2564 feet.

EXAMPLE 3. A river having a hydraulic slope less than ·001 has its section obstructed by the piers and abutments of a bridge to the extent of one-fifth, the normal velocity being 3·5 feet per second, what is the rise caused by the bridge?

By Part 3, Table XI., the rise will be ·12 feet.

N.B.—For rivers having steeper gradients, apply a correction according to the formula given in the text.

TABLE XI.

TABLE OF EQUIVALENTS.

PART 1.—Equivalent supply from total quantities.
2.—Equivalent discharges.
3.—Equivalent velocities.
4.—Equal discharging channels.
5.—Conversion tables for English measures.
6.—Conversion tables for metrical measures.

TABLE XI.—Part 1.

Equivalent Supply.

Continuous supply in cubic feet per second into total quantities, a
vice versâ.

Total quantity in cubic feet.	Continuous supply in cubic feet per second.					
	For 2 months.	For 3 months.	For 6 months.	For 8 months.	For 9 months.	For 12 months.
315 360	·06	·04	·02	·015	·013	·01
630 720	·12	·08	·04	·030	·026	·02
946 080	·18	·12	·06	·045	·040	·03
1 261 440	·24	·16	·08	·060	·053	·04
1 576 800	·30	·20	·10	·075	·066	·05
1 892 160	·36	·24	·12	·090	·080	·06
2 207 520	·42	·28	·14	·105	·093	·07
2 522 880	·48	·32	·16	·120	·106	·08
2 838 240	·54	·36	·18	·135	·120	·09
1 million	·1903	·1268	·0634	·0476	·0423	·031710
2 millions	·3805	·2537	·1268	·0851	·0846	·063420
3 "	·5708	·3805	·1903	·1427	·1268	·095129
4 "	·7610	·5074	·2537	·1902	·1691	·126839
5 "	·9513	·6342	·3171	·2378	·2114	·158549
6 "	1·1416	·7610	·3805	·2854	·2537	·190259
7 "	1·3318	·8879	·4439	·3119	·2960	·221969
8 "	1·5221	1·0147	·5074	·3405	·3382	·253678
9 "	1·7123	1·1416	·5708	·4280	·3805	·285388
10 "	1·9026	1·2684	·6342	·4756	·4228	·317098

[redacted]

TABLE XI.—Part 1—*continued.*

Equivalent Supply.

ous supply in cubic feet per second throughout a month that equivalent to a certain number of waterings in a month.

ts at ter- to re.	At 30 waterings per month.	At 15 waterings per month.	At 10 waterings per month.	At 4 waterings per month.	At 2 waterings per month.	At 1 watering per month.
feet.	Monthly supply in cubic feet per second.					
0	·1157	·0579	·0386	·0154	·0077	·0039
0	·1041	·0520	·0347	·0139	·0069	·0035
0	·0926	·0463	·0309	·0123	·0062	·0031
0	·0810	·0405	·0271	·0108	·0054	·0027
0	·0694	·0347	·0231	·0092	·0046	·0023
0	·0579	·0289	·0193	·0077	·0039	·0019
0	·0463	·0231	·0154	·0062	·0031	·0015
0	·0347	·0173	·0116	·0046	·0023	·0011
0	·0231	·0116	·0077	·0031	·0015	·0008
0	·0116	·0058	·0039	·0015	·0008	·0004
0	·1	·050	·033	·013	·0066	·0033
6	·09	·045	·030	·012	·0060	·0030
2	·08	·040	·027	·011	·0054	·0027
8	·07	·035	023	·009	0046	·0023
4	·06	·030	·020	·008	·0040	·0020
0	·05	·025	·016	·006	·0032	·0016
6	·04	·020	·013	·005	·0026	·0013
2	·03	·015	·010	·004	·0020	.0010
8	·02	·010	·007	·003	·0014	·0007
4	·01	·005	·003	·001	·0007	·0003

N.B.—In this table a month of 30 days is assumed.

[redacted]

TABLE XI.—Part 2—*continued.*

Equivalent Discharges.

per second, per minute, and per day, into Cubic feet per second, per minute, and per day.

cond.		Per minute.		Per day of 24 hours.	
	Cubic ft.	Gallons.	Cubic ft.	Gallons.	Cubic feet.
	·01	6	·96	8640	1385
	·03	12	1·92	17280	2772
	·05	18	2·88	25920	4158
	·06	24	3·84	34360	5543
	·08	30	4·80	43200	6929
	·09	36	5·76	51840	8315
	·11	42	6·72	60480	9701
	·13	48	7·68	69120	11087
	·14	54	8·64	77760	12473
	·16	60	9·62	86400	13858
5	·03	10	1·60	14400	2310
3	·05	20	3·21	28800	4619
	·08	30	4·81	43200	6929
5	·11	40	6·42	57600	9239
3	·14	50	8·02	72000	11549
	·16	60	9·62	86400	13858
5	·19	70	11·23	100800	16168
3	·21	80	12·83	115200	18478
	·24	90	14·44	129600	20788
5	·26	100	16·04	144000	23097
	·186	69·4	111·4	100000	16040
	·371	115·7	222·8	200000	32079
	·557	208·3	334·2	300000	48119
	·742	277·7	445·6	400000	64159
	·928	346·8	556·9	500000	80199
	1·114	416·6	667·3	600000	96239
	1·299	486·	779·7	700000	112278
	1·485	555·5	891·1	800000	128318
	1·670	624·0	1002·5	900000	144358
	1·856	694·4	1113·9	1 million	160398

TABLE XI.—Part 3.

Equivalent Velocities and Heads for Natural Velocities.

Feet per second.	Feet per minute.	Miles per hour.	Corresdg. head of water.	Feet per second.	Feet per minute.	Miles per hour.	Co hea wat
1	60	·6818	·016	3·5	210	2·3866	·19
1·1	66	·7500	·019	3·6	216		·20
1·2	72	·8181	·023	3·7	222		·21
1·3	78	·8863	·026	3·8	228		·22
1·4	84	·9545	·031	3·9	234		·23
1·5	90	1·0238	·035	4	240	2·7264	·25
1·6	96		·040	4·1	246		·26
1·7	102		·045	4·2	252		·27
1·8	108		·051	4·3	258		·29
1·9	114		·056	4·4	264		·30
2	120	1·364	·063	4·5	270	3·0672	·31
2·1	126		·069	4·6	276		·33
2·2	132		·076	4·7	282		·34
2·3	138		·083	4·8	288		·3
2·4	144		·092	4·9	294		·37
2·5	150	1·706	·098	5	300	3·4091	·3
2·6	156		·103	5·1	306		·4
2·7	162		·115	5·2	312		·4
2·8	168		·124	5·3	318		·4
2·9	174		·131	5·4	324		·4
3	180	2·048	·141	5·5	330	3·7500	·4
3·1	186		·151	5·6	336		·4
3·2	192		·160	5·7	342		·5
3·3	198		·170	5·8	348		·5
3·4	204		·180	5·9	354		·5
3·5	210	2·3866	·191	6·	360	4·089	·5

[redacted]

TABLE XI.—Part 3—*continued.*

Feet per second.	Feet per minute.	Miles per hour.	Corrsdg. head of water.	Feet per second.	Feet per minute.	Miles per hour.	Corrsdg. head of water.
6·1	366		·581	9·2	552		1·324
6·2	372		·603	9·4	564		1·380
6·3	378		·620	9·6	576		1·460
6·4	384		·640	9·8	588		1·500
6·5	390	4·2045	·660	10	600	6·818	1·564
6·6	396		·680	10·2	612		1·624
6·7	402		·701	10·4	624		1·644
6·8	408		·720	10·6	636		1·764
6·9	414		·744	10·8	648		1·836
7	420	4·771	·766	11	660	7·500	1·892
7·1	426		·787	11·2	672		1·930
7·2	432		·816	11·4	684		2·032
7·3	438		·832	11·6	696		2·092
7·4	444		·856	11·8	708		2·176
7·5	450	5·1136	·879	12	720	8·1727	2·252
7·9	456		·896	13	780	8·8636	2·6531
7·7	462		·926	14	840	9·5454	3·0625
7·8	468		·952	15	900	10·2272	3·5156
7·9	474		·975	16	960	10·9090	4
8	480	5·453	1	17	1020	11·5909	4·5156
8·2	492		1·052	18	1080	12·2727	5·0625
8·4	504		1·100	19	1140	12·9545	5·6406
8·6	516		1·146	20	1200	13·6363	6·25
8·8	528		1·212	30	1800	20·4545	14·0625
9	540	6·1363	1·265	40	2400	27·2727	25
				50	3000	34·0909	39·062

TABLE XI.—Part 4.

Table of equal-discharging Trapezoidal Channel-Sections (from Stoddard).

Giving depths corresponding to mean widths for any unit;—Slopes 0 to 1 to 2 to 1.

Mean Widths.

70	60	50	40	35	30	25	20	15	10
Relative Depths.									
·25	·27	·30	·35	·40	·45	·52	·58	·71	·98
·50	·55	·62	·73	·80	·89	1·02	1·19	1·48	2·04
·75	·82	·94	1·10	1·20	1·35	1·56	1·82	2·28	3·22
1	1·10	1·26	1·48	1·62	1·81	2·10	2·46	3·11	4·50
1·25	1·39	1·58	1·86	2·04	2·28	2·65	[illegible]	3·98	5·89
1·50	1·67	1·90	2·24	2·46	2·75	3·20	3·80	4·88	7·31
1·75	1·95	2·22	2·62	2·88	3·23	3·75	4·50	5·80	8·86
2	2·23	2·54	3	3·31	3·72	4·32	5·22	6·78	10·50
2·25	2·51	2·86	3·38	3·73	4·21	4·91	5·95	7·81	12·25
2·50	2·79	3·18	3·76	4·16	4·70	5·50	6·68	8·84	14
2·75	3·07	3·51	4·15	4·60	5·21	6·09	7·45	9·93	15·84
3	3·35	3·84	4·54	5·04	5·72	6·69	8·22	11·03	17·68
3·50	3·91	4·5	5·33	5·95	6·75	7·93	9·82	13·32	21·68
4·	4·48	5·14	6·13	6·85	7·81	9·21	11·48	15·75	26
4·50	5·05	5·79	6·95	7·75	8·90	10·50	13·19	18·22	30·36
5	5·62	6·45	7·75	8·66	10	11·79	14·96	20·80	35
5·50	6·18	7·12	8·57	9·62	11·10	13·24	16·77	23·47	39·81

Data of channels of equal discharge (friction-co-officient $f = \cdot 03$) for canals in moderately good order in earth with side slopes of 1½ to 1. S per 1000 is the fall in 1000; D the depth; B the bottom breadth; V mean velocity.

Discharge in cubic feet per second.													
10	S per 1000	0·1	0·5	1·0	1·5	2·0	2·5	3·0	·1	·2	·3	·4	·05
	D	1·31	1·31	1·31	1·31	1·31	1·31	1·31	1·97	1·97	1·97	1·97	2·62
	B	14·60	5·91	3·95	2·83	2·53	2·16	1·87	6·41	4·03	2·95	2·23	4·92
	V	·464	·965	1·282	1·502	1·680	1·827	1·955	·541	·728	·855	·945	·397
100	S per 1000	1·4	3·	·5	1·0	1·6	2·4	3·0	·1	·5	1·0	·1	·05
	D	1·97	1·97	2·62	2·62	2·62	2·62	2·62	3·28	3·28	3·28	3·94	4·59
	B	17·45	11·78	17·85	12·20	9·35	7·32	6·38	27·38	11·32	7·28	19·46	20·57
	V	2·490	3·458	1·748	2·359	2·864	3·373	3·704	·942	1·875	2·490	1·012	·794
1000	S per 1000	1·2	0·5	0·2	0·3	0·4	0·5	0·7	1·0	·1	·2	·1	·05
	D	4·59	5·25	5·91	5·91	5·91	5·91	5·91	5·91	6·56	6·56	7·22	7·87
	B	44·19	54·79	70·77	57·81	49·74	45·97	36·98	30·42	82·35	58·43	69·52	82·32
	V	4·255	3·038	2·129	2·543	2·890	3·195	3·691	4·311	1·653	2·231	1·722	1·358
10000	S per 1000	1·8	1·0	·7	·4	·2	·1	·02	·03	·05	·02	·03	·02
	D	8·53	9·19	9·84	11·48	13·12	14·76	16·40	16·40	16 40	18·04	18·04	19·69
	B	129·46	153·94	172·15	165·91	184·71	206·83	325·20	279·56	229·69	274·12	235·96	233·96
	V	8·238	6·489	5·584	4·751	3·730	2·956	1·742	2·001	2·892	1·837	2·106	1·929

[redacted]

Measures of Capacity.

Units.	Cub. feet into cub. metres.	Gallons into litres.	Bushels into hectolitres.	Units.	Cub. metres into cub. feet.	Litres into gallons.	Hectolitres into bushels.
1	·028315	4·5435	·3635	1	35·317	·22010	2·751
2	·056630	9·0879	·7270	2	70·633	·44019	5·502
3	·084945	13·6304	1·0904	3	105·950	·66029	8·253
4	·113260	18·1738	1·4539	4	141·266	·88039	11·004
5	·141575	22·7173	1·8174	5	176·583	1·10048	13·756
6	·169890	27·2607	2·1808	6	211·900	1·32058	16·506
7	·198205	31·8042	2·5445	7	247·216	1·54068	19·257
8	·226520	36·3476	2·9078	8	282·533	1·76077	22·008
9	·254835	40·8911	3·2712	9	317·849	1·98087	24·759
10	·283150	45·4345	3·6348	10	353·166	2·20097	27·512

Weights.

Units.	Grains troy into grammes.	Pounds into kilogrammes.	Tons into quintals.	Units.	Grammes into grains troy.	Kilogrammes into pounds.	Quintals into tons.
1	·0648	·4535	10·159	1	15·434	2·2048	·0984
2	·1295	·9070	20·318	2	30·868	4·4096	·1969
3	·1943	1·3605	30·477	3	46·302	6·6144	·2953
4	·2591	1·8140	40·636	4	61·736	8·8192	·3938
5	·3239	2·2675	50·797	5	77·170	11·0240	·4922
6	·3886	2·7210	60·954	6	92·604	13·2288	·5907
7	·4534	3·1745	71·113	7	108·038	15·4336	·6891
8	·5182	3·6280	81·272	8	123·472	17·6384	·7876
9	·5829	4·0815	91·436	9	138·906	19·8432	·8856
10	·6478	4·5350	101·595	10	154·340	22·0480	·9844

TABLE XI.—Part 6—*continued.*

Measures of Water Supply.

A watering in cubic feet per acre of		A watering in cubic metres per hectare of	A watering in cubic metres per hectare of		A watering in cubic feet per acre of
10000	=	700	1000	=	14284
9000	=	630	900	=	12852
8000	=	560	800	=	11424
7000	=	490	700	=	9996
6000	=	420	600	=	8568
5000	=	350	500	=	7142
4000	=	280	400	=	5712
3000	=	210	300	=	4284
2000	=	140	200	=	2856
1000	=	70	100	=	1428

A supply in litres per second per hectare of		A supply in cubic feet per second per acre of	A supply in cubic feet per second per acre of		A supply in litres per second per hectare of
2·00	=	·02856	·03	=	2·10018
1·50	=	·02142	·02	=	1·40012
1·00	=	·01428	·01	=	·70006
·75	=	·01021	·0075	=	·52004
·5	=	·00714	·0050	=	·35003
·25	=	·00357	·0025	=	·17501

1 litre per second per hectare = a duty of 70 acres per cubic foot per second.

·01 cubic feet per second per acre = a duty of 100 acres per cubic foot per second.

A hectare is equal to 10 000 square metres.

A litre is equal to $\frac{1}{1000}$ of a cubic metre.

TABLE XI.—PART 6—*continued*

Measures of Pressure.

quare inch into kilogrammes er square centimetre.		Kilogrammes per square centimetre into lbs. per square inch.	
	·0703	1	14·237
	·1406	2	28·475
	·2109	3	42·713
	·2812	4	56·950
	·3515	5	71·187
	·4218	6	85·426
	·4921	7	99·663
	·5624	8	113·901
	·6327	9	128·138
	·7023	10	142·375

Measures of Heat.

Fahrenheit.	Centigrade.	Fahrenheit.	Centigrade.	Fahrenheit.	Centigrade.	Fahrenheit.
32°	20°	68°	25°	77°	30°	86°
36·5	20·5	68·9	25·5	77·9	30·5	86·9
41	21	69·8	26	78·8	31	87·8
45·5	21·5	70·7	26·5	79·7	31·5	88·7
50	22	71·6	27	80·6	32	89·6
54·5	22·5	72·5	27·5	81·5	32·5	90·5
59	23	73·4	28	82·4	33	91·4
63·5	23·5	74·3	28·5	83·3	33·5	92·3
68	24	75·2	29	84·2	34	93·2
	24·5	76·1	29·5	85·1	34·5	94·1
95°	40°	104°	45°	113°	50°	122°
95·9	40·5	104·9	45·5	113·9	55	131
96·8	41	105·8	46	114·8	60	140
97·7	41·5	106·7	46·5	115·7	65	149
98·6	42	107·6	47	116·6	70	158
99·5	42·5	108·5	47·5	117·5	75	167
100·4	43	109·4	48	118·4	80	176
101·3	43·5	110·3	48·5	119·3	85	185
102·2	44	111·2	49	120·2	90	194
103·1	44·5	112·1	49·5	121·1	100	212°

TABLE XII.

PART 1.—Co-efficients of fluid friction.
,, 2.—Co-efficients of flood discharge from catchment areas.
,, 3.—Co-efficients of discharge for rivers, canals, and pipes.
,, 4.—Co-efficients of discharge for orifices.
,, 5.—Co-efficients of discharge for overfalls.
,, 6.—Hydraulic memoranda.
,, 7.—Useful numbers, powers, roots, &c.

[redacted]

TABLE XII.—PART 1.

efficients of fluid friction, being the values of *f* in the formula given in the text.

(D'Arcy, Bazin, Ganguillet, and Kutter.)

(From the "Cultur-Ingénieur," 1870.)

General values.

—Well planed plank.

—Very smooth surfaces, plasters in cement; assumed to be applicable also to enamelled and glazed pipes.

—Plaster in cement, with one-third sand.

—Unplaned plank.

—Brickwork and cut stone; assumed to apply also to metal and earthenware pipes under ordinary conditions, but not new.

7—Rubble masonry.

—Canals with bed and banks of very firm gravel.

5—Rivers and Canals in Earth, in perfect order and regimen, and perfectly free from stones and weeds.

0—Rivers and Canals in Earth, in moderately good order and regimen, having stones and weeds occasionally.

5—Rivers and Canals in Earth, in bad order and regimen, having stones and weeds in great quantity.

Local values.

·019—The Marseilles Canal.

·022—Rigoles de Grosbois.

·023—Tauber Alpbachschale, Rhine.

·024—Linth canal. Hübengraben. Hill-streams.

·025—Jard canal. Seine. Neva.

·026—Seine. Haine. Rhine. Speierbach.

·027—Mississippi. Rhine.

·028—Saone, Salzach.

·029—Danube in Hungary.

·030—Rigoles de Chazilly.

·031—Limat, Zurich.

·033—Maras.

·035—Simme.

TABLE XII.—PART 2.

Co-efficients of flood discharge from catchment are

FOR the formula in Table IV., Part 1, also given in the text

$$Q = n \times 100 (N)^{\frac{3}{4}}$$

The value of this co-efficient (n) can be determined and made of within local limits only, as it depends on the average rai local downpour, the evaporation, the quality, inclination, and di sition of the surface of the area under consideration; it has hithert been determined for very few districts, and not sufficiently satisfa torily for some of those. In some cases, unfortunately, doubtful fl marks have been used to obtain the flood gradient, and the velociti calculated according to very varied formulæ; in others, the obstru tions caused by bridges and embankments have vitiated all the ba of calculation of discharge.

	Values of
For very large Indian rivers near their mouths ...	·08 to
For Oudh generally	1 to
The Madras Presidency, the whole Cavery The Godavery, Kistna, Tumbaddra, Pennair, Vigay } about	2·
The Chittaur, Palaur, Manjilanthi, Varhasanthi below ...	5·
For the Kanhan River, Central Provinces, according to the highest flood yet known, less than	5·
For Bengal and Bahar, rainfall 2 to 4 feet—Col. Dickens gives a co-efficient of	8·25
The Upper Cavery, Tambrapurni, Gadanamatti ...	12, 1 and
For some rivers in Berar and the Central Provinces, according to calculated velocities only	16· to

Some further data for Indian rivers will be found in the Statistics.

For vals of S.	For values of R in feet.											
	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·	1·25	1·50
·0001	·72	·92	1·03	1·13	1·22	1·26	1·29	1·35	1·38	1·41	1·48	1·54
·0002	·77	·98	1·08	1·17	1·27	1·31	1·35	1·38	1·42	1·44	1·51	1·56
·0003	·81	1·00	1·11	1·19	1·29	1·32	1·36	1·40	1·43	1·44	1·52	1·57
·0004	·82	1·01	1·12	1·21	1·29	1·33	1·37	1·41	1·44	1·46	1·52	1·57
·0005	·82	1·02	1·12	1·22	1·30	1·34	1·38	1·41	1·44	1·47	1·53	1·58
·0008	·83	1·03	1·13	1·22	1·31	1·35	1·38	1·42	1·45	1·48	1·53	1·58
·001 and steeper falls.	·84	1·04	1·14	1·23	1·31	1·35	1·38	1·43	1·46	1·49	1·54	1·58

Applying the above to glazed and enamelled pipes, having gradients steeper than ·001, and neglecting very small pipes, for which experimental data are wanting, the co-efficients will be—

For diameters of	5 inches and under,	6″,	7″,	8″,	9″,	10″,	12″
Co-efficients	·84	·87,	·91,	·95,	·99,	1·04,	1·09.

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces with a frictional co-efficient $f = \cdot 01$, suitable to very smooth plastered channels in cement, and to enamelled and glazed pipes.

For values of S.	For values of R in feet.											
	1·75	2·0	2·5	3·0	3·5	4·0	4·5	5	6	8	10	12
·0001	1·58	1·62	1·68	1·74	1·78	1·82	1·85	1·87	1·91	1·96	2·02	2·06
·0002	1·60	1·63	1·70	1·74	1·77	1·81	1·83	1·85	1·89	1·94	1·98	2·01
·0003	1·61	1·64	1·70	1·74	1·77	1·80	1·83	1·85	1·88	1·93	1·97	1·99
·0004	1·61	1·65	1·70	1·74	1·77	1·80	1·83	1·85	1·88	1·93	1·97	1·99
·0005	1·62	1·65	1·70	1·74	1·77	1·80	1·82	1·84	1·87	1·92	1·96	1·98
·0008	1·63	1·65	1·70	1·74	1·77	1·80	1·82	1·84	1·87	1·92	1·96	1·98
·001 and steeper falls.	1·63	1·65	1·70	1·74	1·77	1·80	1·82	1·84	1·86	1·91	1·95	1·97

For values of S.	For values of R in feet.											
	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0	1·25	1·50
·0001	·50	·66	·74	·84	·90	·94	·96	1·00	1·04	1·05	1·12	1·17
·0002	·55	·71	·79	·87	·94	·97	1·00	1·04	1·07	1·09	1·15	1·19
·0003	·58	·72	·81	·89	·96	·99	1·01	1·05	1·08	1·10	1·15	1·19
·0004	·59	·73	·82	·89	·96	·99	1·02	1·06	1·08	1·10	1·15	1·20
·0005	·60	·74	·83	·90	·97	1·00	1·03	1·07	1·09	1·10	1·16	1·20
·0006	·60	·75	·83	·91	·97	1·01	1·03	1·07	1·09	1·11	1·17	1·20
·001 and steeper falls.	·61	·75	·83	·92	·98	1·01	1·04	1·07	1·10	1·12	1·17	1·21

Applying the above to ordinary metal pipes, having gradients steeper than ·001, and neglecting very small pipes for which experimental data are wanting, the co-efficients will be—

For diameters of	5 inches and under,	6″,	7″,	8″,	9″,	10″,	12
Co-efficients	·61	·63,	·66,	·69,	·72,	·75,	·79.

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot 013$—*continued.*

(Kutter.)

For values of S.	For values of R in feet.											
	1·75	2·	2·5	3·	3·5	4·	4·5	5·	6·	8·	10	12
·0001	1·22	1·24	1·30	1·35	1·38	1·41	1·44	[illegible]	[illegible]	1·57	1·61	1·64
·0002	1·23	1·26	1·30	1·35	1·38	1·41	1·43	[illegible]	[illegible]	·53	1·57	1·60
·0003	1·23	1·27	1·31	1·35	1·38	1·40	1·43	1·44	1·47	1·52	1·56	1·57
·0004	1·23	1·27	1·31	1·35	1·38	1·40	1·42	1·44	1·47	1·52	1·55	1·57
·0005	1·24	1·27	1·31	1·35	1·38	1·40	1·42	1·43	1·47	1·51	1·55	1·57
·0008	1·24	1·28	1·31	1·35	1·38	1·39	1·42	1·43	1·47	1·51	1·54	1·56
·001 and steeper falls.	1·24	1·28	1·31	1·35	1·38	1·39	1·41	1·43	1·46	1·50	1·53	1·56

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot 017$, suitable to channels or aqueducts in rubble. (Kutter.)

For values of S.	For values of R in feet.												
	·1	·5	·75	1	2	3	4	5	6	7	8	9	10
.0001	·35	·65	·72	·78	·93	1·03	1·09	1·13	1·17	1·20	1·22	1·25	1·27
·0002	·38	·67	·75	·80	·95	1·03	1·08	1·12	1·16	1·18	1·20	1·22	1·23
·0003	·39	·68	·76	·81	·95	1·03	1·08	1·12	1·15	1·17	1·19	1·21	1·22
·0004	·40	·69	·76	·82	·95	1·03	1·08	1·11	1·15	1·17	1·19	1·20	1·22
·0005	·40	·70	·77	·82	·95	1·03	1·07	1·11	1·14	1·17	1·18	1 20	1·21
·0008	·41	·71	·77	·83	·96	1·03	1·07	1·11	1·14	1·17	1·18	1·19	1·21
·001 and steeper falls.	·41	·71	·78	·83	·96	1·03	1·07	1·11	1·13	1·16	1·17	1·19	1·21

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot025$, suitable to canals and rivers with earthen beds in perfect order and regimen, and perfectly free from stones and weeds. (Kutter.)

For values of S.	For values of R in feet.												
	·5	·75	1	1·5	2	2·5	3	3·5	4·	4·5	5·	6	7
·00005					·62	·67	·70	·74	·77	·79	·81	·85	·88
·00007					·63	·67	·71	·73	·76	·78	·80	·84	·87
·0001	·40	·47	·52	·58	·64	·68	·71	[illegible]	·76	·77	·79	·83	·86
·0002	·43	·49	·53	·60	·64	·68	·71	·73	·75	·77	·79	·81	·84
·0003	·44	·50	·54	·60	·65	·68	·71	·73	·75	·77	·78	·81	·83
·0004	·44	·50	·54	·60	·65	·68	·71	·73	·75	·76	·78	·80	·83
·0005	·44	·50	·54	·60	·65	·68	·71	·73	·75	·76	·78	·80	·82
·0008	·45	·51	·55	·61	·65	·69	·71	·73	·75	·76	·78	·80	·82
·001 and steeper	·45	·51	·55	·61	·65	·69	·71	·73	·75	·76	·78	·80	·82

TABLE XII.—Part 3—continued.

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot025$, suitable to canals and ri earthen beds in perfect order and regimen, and perfectly free from stones and weeds—continued. (Kutter.)

For values of S.	For values of R in feet.											
	8	9	10	11	12	13	14	15	16	17	18	19
·00002					1·10	1·10	1·15	1·17	1·19	1·21	1·23	1·25
·00003					1·05	1·07	1·08	1·10	1·12	1·14	1·15	1·17
·00005	·91	·93	·96	·98	1·00	1·01	1·03	1·04	1·06	1·08	1·09	1·09
·00007	·89	·91	·93	·95	·97	·98	·99	1·00	1·01	1·03	1·04	1·05
·0001	·88	·90	·91	·93	·94	·95	·97	·97	·99	1·00	1·00	1·01
·0002	·85	·87	·89	·90	·91	·92	·93	·94	·95	·95	·96	·97
·0003	·85	·87	·88	·89	·90	·91	·92	·92	·93	·94	·95	·95
·0004	·85	·86	·87	·88	·89	·91	·91	·92	·93	·93	·94	·94
·0005	·84	·86	·87	·88	·89	·90	·90	·91	·92	·93	·93	·94
·0008	·84	·85	·87	·87	·88	·89	·90	·91	·91	·92	·92	·93
·001 and steeper falls.	·84	·85	·86	·87	·88	·89	·90	·91	·91	·92	·92	·93

(Kutter.)

For values of s.	For values of R in feet.												
	8	9	10	11	12	13	14	15	16	17	18	19	20
·00002					·94	·96	·99	1·01	1·03	1·05	1·06	1·08	1·09
·00003					·90	·92	·94	·96	·97	·99	1·00	1·02	1·03
·00005	·78	·80	·82	·84	·85	·87	·88	·90	·91	·92	·94	·95	·95
.00007	·76	·78	·80	·82	·83	·84	·86	·87	·88	·89	·90	·91	·92
·0001	·74	·76	·78	·79	·81	·82	·83	·84	·85	·86	·87	·88	·88
·0002	·73	·74	·76	·77	·78	·79	·80	·81	·81	·82	·83	·85	·85
·0003	·72	·74	·75	·76	·77	·78	·79	·80	·80	·81	·82	·82	·83
·0004	·72	·73	·74	·75	·77	·77	·78	·79	·80	·80	·81	·82	·82
0005	·72	·73	·74	·75	·76	·77	·78	·79	·79	·80	·80	·81	·82
·0008	·71	·73	·73	·74	·75	·76	·77	·78	·78	·79	·79	·80	·80
·001 and steeper falls	·71	·73	·78	·74	·75	·76	·77	·78	·78	·79	·79	·80	·80

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient f = ·035, suitable to rivers and canals with earthen beds in bad order and regimen, having stones and weeds in great quantities. (Kutter.)

For values of S.	For values of R in feet.											
	·5	·75	1	2	3	4	5	6	7	8	9	10
·00005				·44	·50	·55	·59	·62	·65	·68	·70	·72
·00007				·44	·50	·55	·58	[illegible]	·64	·66	·68	·70
·0001	·27	·32	·35	·45	·50	·55	·58	[illegible]	[illegible]	·65	·67	·69
·0002	·28	·33	·36	·45	·51	·54	·57	·60	·62	·64	·65	·66
·0003	·29	·33	·37	·45	·51	·54	·57	·59	·61	·63	·64	·66
·0004	·29	·34	·37	·45	·51	·54	·57	·59	·61	·63	·64	·65
·0005	·29	·34	·37	·46	·51	·54	·57	·59	·61	·62	·64	·65
·0008	·30	·35	·37	·46	·51	·54	·57	·59	·61	·62	·63	·64
·001 and	·30	·35	·37	·46	·51	·54	·57	·59	·61	·62	·63	·64

[redacted]

TABLE XII.—Part 4.

fficients of Discharge for Orifices, being values of m for the formula in Table IX., and given in the Text.

$$V = m \times 8{\cdot}025 \sqrt{H}$$

d	According to Experiment.	
	·572	Rectangular, length 7 depth, (L 7 D); *see* next page.
	·709	
	·62	Orifices generally.
	·66	Sluices without side walls.
	·7	Canal lock gates and dock gates.
7	·62	Undershot wheel gates.
	·8	Velocity of approach in a channel.
	·83	Sluices in lock gates.
	·84	Large vertical pipes.
	·9	Narrow bridge openings.
	·94	Large sluices.
	·96	Wide openings from reservoirs.
	·96	Wide bridge openings.
	·96	Orifices with converging mouth-pieces.
	1·	Large orifices with diverging mouth-pieces.
	1·3	Attached diverging mill channels.

odification of the co-efficient m, so as to include the effect due to city of approach;

Let h = head due to this velocity only,

$$\text{then } m^1 = m \sqrt{1 + \frac{h}{H}}$$

m^1 is the new co-efficient to be used.

TABLE XII.—Part 4—*continued.*

Co-efficients of Discharge for Orifices—*continued.*

Table of co-efficients of Velocit[illegible] Discharge for Rectangular Orific
when the depth (D) is less [illegible] he width (W) for a head (H).

$\frac{H}{W}$	$\frac{D}{W}$ = ·1	$\frac{D}{W}$ = ·5	5	$\frac{D}{W}$ = ·15	$\frac{D}{W}$ = ·1	$\frac{D}{W}$ = ·05
			es of M.			
·05						·709
·10					·660	·698
·15				·638	·660	[illegible]
·20			·612	·640	·659	·685
·25			·617	·640	·659	·682
·30			·622	·640	·658	·676
·40		·600	·620	·639	·657	·671
·50		·605	·628	·638	·655	·667
·60	·572	·609	·630	·637	·654	·664
·75	·585	·611	·631	·635	·653	·660
1·00	·592	·613	·634	·634	·650	·655
1·50	·598	·616	·632	.632	·645	·650
2·00	·400	·617	·631	·631	·642	·647
2·50	·602	·617	·631	·630	·640	·643
3·50	·604	·616	·629	·629	·637	·638
4·00	·605	·615	·627	·627	[illegible]	·627
6·00	·604	·613	·623	·623	·625	[illegible]
8·00	·602	·611	·619	·619	·618	·616
10·00	·601	·607	·613	·613	·613	[illegible]

The above was deduced by Rankine from results of experiments Poncelet and Lesbros.

N.B.—When H 7 3 D, the centre of figure may be considered centre of motion.

TABLE XII.—Part 5.

ficients of Discharge for Overfalls, being values of *m* for the formula applied in Table IX., and given in the Text.

$$V = \tfrac{2}{3}\, m \times 8{\cdot}025 \sqrt{H}$$

1 = length of weir sill : L = length of dam, or breadth of el : H = head on sill : D = depth of notch.

By Experimentalists.	
·5	Broad-crested or flat-topped dams Dams with a channel attached
·595 ·662	Weirs with 1-inch crests when $1 =$ or $> \frac{L}{4}$; the exact value of *m* being $= \cdot 57 \times \frac{1}{10L}$.
·6	Overfalls when $1 > \frac{L}{4}$ and $< \frac{L}{3}$ V-shaped notch, when $1 = \frac{D}{2}$
·26	V-shaped notch, when $1 = \frac{D}{4}$
·552	Weirs when $1 = L$, and $H > \frac{1}{3}$ height of the barrier; in this case the velocity of approach must be considered in addition.
·666	Weirs generally when $1 = L$ and $H < \frac{1}{3}$ the height of the barrier.

modify the co-efficient *m* so as to include the effect due to y of approach,

h = head due to velocity of approach only ;—

$$n^1 = m\left\{ \left(1 \times \frac{h}{H}\right)^{\frac{3}{2}} - \left(\frac{h}{H}\right)^{\frac{3}{2}} \right\}$$

^{1} is the new co-efficient to be used.

---

using Table VIII. for overfalls, always diminish the velocity of discharge there y one-third ; this alone admits of the use of the same table for discharges both es and overfalls.

TABLE XII.—Part 6.

Hydraulic Memoranda.

Measures.

Feet × ·015 = Gunter's chains.
Feet × ·00019 = Miles.
Square feet × ·11 = Square yards.
Square feet × ·00[illegible] = Acres.
Cubic feet × 6·23 = Gallons.
Cubic feet × ·779 = Bushels.
Cubic feet × ·037 = Cubic yards.

Rainfall.

Feet of downpour × 193600· = cubic feet per square mile.
Feet of downpour × 302·5 = cubic feet per acre.

Drainage areas.

The drainage from 1 square mile collecting 1 foot yearly } will irrigate 176 acres a duty of 200 acres, will su 47,580 inhabitants at a of 10 gallons daily, will ·8833 cubic feet per se throughout the year.

Velocities.

Feet per second × ·68 give miles per hour.
Feet per second × 60 give feet per minute.
Feet per second × 20 give yards per minute.
Feet per second × 1200 give yards per hour.

Discharges.

Cub. feet per sec. × 2·2 give cubic yards per minute.
Cub. feet per sec. × 133 give cubic yards per hour.
Cub. feet per sec. × 3200 give cubic yards per day.
Cub. feet per sec. × 6¼ give gallons per second.
Cub. feet per sec. × 375 give gallons per minute.
Cub. feet per sec. × 22 give thousands of gallons per ho
Cub. feet per sec. × 500 give thousands of gallons per d
Cub. feet per sec. × 2400 give tons per day.

[redacted]

TABLE XII.—Part 6—*continued.*

ic feet.		Gallons.	
1·	=	6·232	and weighs 62·32 lbs.
·1605	=	1	and weighs 10 lbs.
1·8	=	11·2	~~and~~ weighs 1 cwt.
35·943	=	224	and weighs 1 ton.
1 cubic inch	=	·0036	and weighs ·0361 lbs.

luid ounce weighs 437·5 grains.
Troy ounce measures 8 fluid ounces, 46 minims.
Avoirdupois ounce measures 8 fluid ounces.
lb. Troy =5760 grains =6319·54 minims of water.
gallon =76800 minims =70000 grs. of distilled water.
lb. Avoir. =7000 grains.

All comparisons between measures of capacity and those of weight re made with distilled water at a specific gravity of 1, temp. 62°.

Pressure.

H = head of water in feet H = P × 2·31.
P = pressure in lbs. per square foot P = H × 62·32.

Horse-power.

HP = 33000 lbs. raised 1 foot in 1 minute.
= 884 tons raised 1 foot in 1 hour.

eoretical HP = ·113 Q × fall in feet.

The drainage of 10 square miles collecting 12″ yearly gives HP for each foot of fall.

For pumping engines of the best class, allow HP = ·142 Q H where = quantity raised in cubic feet per second, H = height in feet.

Mills.—An ordinary mill will grind 1 bushel per HP per hour; r each pair of stones allow 4 HP nominal.

Towage.

The general formula referred to in the text is

$$R = b\,T.\,V^2,$$

where R = the pull on the rope in pounds,
T = the displacement of the barge in tons,
V = the velocity through the water,
b = a co-efficient varying with the form of the barge, from ·109 to ·369.

TABLE XII.—Part 7.

Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area of Circle.	Square.	Square Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
·01	·031	·00008	·0001	·1	·2154	·3981	·00001	·1584	100·	$\bar{2}$·
·015	·047	·00018	·0002	·1225	·2466	·4317	·00003	·1864	66·66	$\bar{2}$·1760913
·02	·063	·0003	0004	·1414	·2714	·4573	·00006	·2140	50·	$\bar{2}$·3010300
·025	·078	·0005	·0006	·1581	·2924	·4782	·00010	·2287	40·	$\bar{2}$·3979400
·03	·094	·0007	·0009	·1732	·3107	·4959	·00017	·2460	33·33	$\bar{2}$·4771213
035	·110	·0010	·0012	·1871	·3271	·5115	·00023	·2616	28·57	$\bar{2}$·5440680
·04	·126	·0013	·0016	·2	·3420	·5253	·00032	·2759	25·	$\bar{2}$·6020600
·045	·141	·0016	·0020	·2121	·3557	·5378	·00043	·2893	22·22	$\bar{2}$·6532125
·05	·157	·0020	·0025	·2236	·3684	·5493	·00056	·3017	20·	$\bar{2}$·6989700
·055	·173	·0024	·0030	·2345	·3803	·5599	·00071	·3134	18·18	$\bar{2}$·7403627
·06	·188	·0028	·0036	·2449	·3915	·5697	·00088	·3245	16·67	$\bar{2}$·7781513
·065	·204	·0033	·0042	·2550	·4021	.5789	·00108	·3351	15·38	$\bar{2}$·8129134
·07	·220	·0038	·0049	·2646	·4121	·5875	·00130	·3452	14·29	$\bar{2}$·8450980
·075	·236	·0044	·0056	·2739	·4217	·5957	·00154	·3548	13·33	$\bar{2}$·8750613
·08	·251	·0050	·0064	·2828	·4309	·6034	·00181	·3641	12·5	$\bar{2}$·9030900
·085	·267	·0057	·0072	·2915	·4397	·6108	·00211	·3731	11·76	$\bar{2}$·9294189
.09	·283	·0064	·0081	·3	·4481	·6178	·00243	·3817	11·11	$\bar{2}$·9542425

TABLE XII.—PART 7—*continued.* Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
·1	·314	·0078	·01	·3162	·4642	·6310	·0032	·3081	10·	1̄·
·15	·471	·0177	·0225	·3873	·5313	·6843	·0087	·4682	6·666	1̄·1760913
·2	·628	·0314	·04	·4472	·5848	·7248	·0179	·5258	5·	1̄·3010300
·25	·785	·0491	·0625	·5	·6300	·7579	·0313	·5743	4·	1̄·3979400
·3	·942	·0706	·09	·5477	·6694	·7860	·0493	·6178	3·333	1̄·4771213
·35	1·100	·0962	·1225	·5916	·7047	·8106	·0769	·6571	2·857	1̄·5440680
·4	1·257	·1256	·1600	·6324	·7368	·8326	·1012	·6931	2·5	1̄·6020600
·45	1·414	·1590	·2025	·6708	·7663	·8524	·1358	·7266	2·222	1̄·6532125
·5	1·571	·1963	·25	·7071	·7937	·8706	·1768	·7579	2·	1̄·6989700
·52	1·634	·2124	·2704	·7211	·8041	·8774	·1950	·7698	1·923	1̄·7160033
·54	1·696	·2290	·2916	·7348	·8143	·8841	·2143	·7816	1·852	1̄·7323938
·56	1·759	·2463	·3136	·7483	·8243	·8905	·2347	·7930	1·786	1̄·7481880
·58	1·822	·2642	·3364	·7616	·8340	·8968	·2562	·8042	1·724	1̄·7634280
·6	1·884	·2827	·36	·7746	·8434	·9029	·2788	·8152	1·667	1̄·7781513
·62	1·948	·3019	·3844	·7874	·8527	·9088	·3027	·8260	1·613	1̄·7923917
·64	2·011	·3217	·4096	·8	·8618	·9146	·3277	·8365	1·563	1̄·8061800
·66	2·073	·3421	·4356	·8124	·8707	·9203	·3539	·8469	1·515	1̄·8195439
·68	2·136	·3632	·4624	·8246	·8794	·9258	·3813	·8570	1·471	1̄·8325089
·7	2·199	·3848	·49	·8366	·8879	·9312	·4100	·8670	1·429	1̄·8450980
·72	2·262	·4072	·5184	·8485	·8963	·9364	·4309	·8769	1·389	1̄·8573325
·74	2·325	·4301	·5476	·8602	·9045	·9416	·4711	·8865	1·351	1̄·8692317
·76	2·388	·4537	·5776	·8718	·9126	·9466	·5036	·8961	1·316	1̄·8808316
·78	2·450	·4778	·6084	·8832	·9205	·9515	·5373	·9054	1·282	1̄·8920946
·8	2·513	·5026	·64	·8944	·9283	·9564	·5724	·9146	1·250	1̄·9030900
·82	2·576	·5281	·6724	·9055	·9360	·9611	·6089	·9237	1·220	1̄·9138139

E XII.—Part 7—*continued.* Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
·84	2·639	·5542	·7056	·9165	·9435	·9657	·6467	·9327	1·190	$\bar{1}$·9242793
·86	2·702	·5809	·7396	·9274	·9510	·9702	·6859	·9415	1·163	$\bar{1}$·9344985
·88	2·765	·6082	·7744	·9381	·9583	·9748	·7265	·9502	1·136	$\bar{1}$·9444827
·9	2·827	·6362	·81	·9487	·9655	·9791	·7684	·9587	1·111	$\bar{1}$·9542425
·92	2·890	·6648	·8464	·9592	·9726	·9834	·8118	·9672	1·087	$\bar{1}$·9637878
·94	2·953	·6940	·8836	·9695	·9796	·9877	·8567	·9756	1·064	$\bar{1}$·9731279
·96	3·016	·7238	·9216	·9798	·9865	·9918	·9030	·9838	1·042	$\bar{1}$·9822712
·98	3·079	·7543	·9604	·9899	·9933	·9960	·9507	·9920	1·020	$\bar{1}$·9912261
1·	3·141	·7854	1·	1·	1·	1·	1.	1·	1·	0·
1·25	3·927	1·2272	1·5625	1·1180	1·0772	1·0456	1·7469	1·0934	·8	0·0969100
1·5	4·712	1·7671	2·25	1·2247	1·1447	1·0845	2·7556	1·1761	·6666	0·1760913
1·75	5·947	2·4053	3·0625	1·3229	1·2051	1·1184	4·0513	1·2509	·5714	0·2430880
2·	6·283	3·1416	4·	1·4142	1·2599	1·1487	5·6569	1·3195	·5	0·3010300
2·25	7·068	3·9761	5·0625	1·5	1·3104	1·1761	7·5937	1·3832	·4444	0·3521825
2·5	7·854	4·9087	6·25	1·5811	1·3572	1·2011	9·8823	1·4427	·4	0·3979400
2·75	8·639	5·9396	7·5625	1·6583	1·4010	1·2242	12·541	1·4988	·3636	0·4393327
3·	9·425	7·0686	9·	1·7321	1·4423	1·2457	15·589	1·5518	·3333	0·4771213
3·25	10·21	8 2058	10·5625	1·8028	1·4812	1·2658	19·041	1·6023	·3077	0·5118834
3·5	10 99	9·6211	12 25	1 8708	1·5183	1·2846	22·918	1·6505	·2857	0·5440680
3·75	11·78	11·0469	14·0625	1·9365	1·5536	1·3026	27·232	1·6967	·2666	0·5740313
4·	12·57	12·5664	16·	2·	1·5874	1·3195	32·	1·7411	·25	0·6020600
4·25	13·35	14·1863	18·0625	2·0616	1·6198	1·3356	37·2361	1·7838	·2353	0·6283889
4·5	14·13	15·9043	20·25	2·1213	1·6510	1·3510	42·9561	1·8251	·2222	0·6532125
4·75	14·92	17·7206	22·5625	2·1794	1·6810	1·3656	49·1781	1·8650	·2105	0·6766936
5·	15·71	19·6350	25·	2·2361	1·7099	1·3804	55·9010	1·9034	·2	0·6989700

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
5·5	17·27	23·7583	30·25	2·3452	1·7652	1·4063	70·94	1·9776	·1818	0·7403627
6·	18·84	28·2744	36·	2·4495	1·8171	1·4310	88·18	2·0477	·1667	0·7781513
6·5	20·42	33·1831	42·25	2·5495	1·8663	1·4541	107·71	2·1143	·1538	0·8129134
7·	21·99	38·4846	49·	2·6458	1·9129	1·4758	129·64	2·1779	·1429	0·8450980
7·5	23·56	44·1787	56·25	2·7386	1·9574	1·4963	154·04	2·2388	·1333	0·8750613
8·	25·13	50·2656	64·	2·8284	2·	1·5157	181·01	2·2974	·1250	0·9030900
8·5	26·70	56·7451	72·25	2·9155	2·0408	1·5342	214·64	2·3538	·1176	0·9294189
9·	28·27	63·6174	81·	3·	2·0801	1·5518	243·	2·4082	·1111	0·9542425
9·5	29·84	70·8823	90·25	3·0822	2·1179	1·5687	278·16	2·4609	·1053	0·9777236
10·	31·41	78·5400	100·	3·1623	2·1544	1·5849	316·23	2·5119	·1	1·
11	34·55	95·033	121	3·3166	2·2239	1·6154	401·31	2·6095	·0909	1·0413927
12	37·69	113·097	144	3·4641	2·2894	1·6437	498·83	2·7019	·0833	1·0791812
13	40·84	132·732	169	3·6056	2·3513	1·6702	609·34	2·7896	·0769	1·1139434
14	43·98	153·938	196	3·7417	2·4101	1·6952	733·36	2·8738	·0714	1·1461280
15	47·12	176·715	225	3·8729	2·4662	1·7188	871·42	2·9543	·0667	1·1760913
16	50·26	201·062	256	4·	2·5198	1·7411	1024·	3·0314	·0625	1·2041200
17	53·40	226·980	289	4·1231	2·5713	1·7623	1191·8	3·1058	·0588	1·2304489
18	56·54	254·469	324	4·2426	2·6207	1·7826	1374·6	3·1777	·0556	1·2552725
19	59·69	283·529	361	4·3589	2·6684	1·8020	1573·5	3·2472	·0526	1·2787536
20	62·83	314·160	400	4·4721	2·7144	1·8206	1788·8	3·3145	·0500	1·3010300
21	65·97	346·361	441	4·5826	2·7589	1·8384	2020·9	3·3798	·0476	1·3222193
22	69·11	380·133	484	4·6904	2·8020	1·8556	2270·11	3·4438	·0454	1·3424227
23	72·25	415·476	529	4·7958	2·8439	1·8722	2537·00	3·5050	·0435	1·3617278
24	75·39	452·390	576	4·8989	2·8845	1·8882	2821·8	3·5652	·0417	1·3802112
25	78·54	490·875	625	5·	2·9240	1·9037	3125·0	3·6239	·0400	1·3979400

TABLE XII.—Part 7—*continued*. Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
26	81·68	530·930	676	5·0990	2·9625	1·9186	3446·9	3·6812	·0385	1·4149733
27	84·82	572·556	729	5·1962	3·	1·9332	3788·0	3·7372	·0370	1·4313638
28	87·96	615·753	784	5·2915	3·0366	1·9473	4148·5	3·7920	·0357	1·4471580
29	91·10	660·521	841	5·3852	3·0723	1·9610	4528·9	3·8455	·0345	1·4623980
30	94·24	706·860	900	5·4772	3 1072	1·9744	4929·5	3·8981	·0333	1·4771213
31	97·38	754·769	961	5·5678	3·1414	1·9873	5350·6	3·9493	·0323	1·4913617
32	100·5	804·249	1024	5·6569	3·1748	2·	5792·6	4·	·0313	1·5051500
33	103·6	855·300	1089	5·7746	3·2075	2·0124	6255·8	4·0495	·0303	1·5185139
34	106·8	907·922	1156	5·8309	3·2396	2·0244	6740·5	4·0982	·0294	1·5314789
35	109·9	962·115	1225	5·9161	3·2711	2·0362	7247·2	4·1460	·0285	1·5440680
36	113·0	1017·87	1296	6·	3·3019	2·0477	7776·0	4·1980	·0278	1·5563025
37	116·2	1075·21	1369	6·0828	3·3322	2·0589	8327·3	4·2392	·0270	1·5682017
38	119·3	1134·11	1444	6·1644	3·3620	2·0699	8901·4	4·2846	·0263	1·5797836
39	122·5	1194·59	1521	6·2449	3·3912	2·0807	9498·6	4·3294	·0256	1·5910646
40	125·6	1256·64	1600	6·3245	3·4200	2·0913	10120	4·3785	·0250	1·6020600
41	128·8	1320·25	1681	6·4031	3·4482	2·1017	10763	4·4169	·0244	1·6127839
42	131·9	1385·44	1764	6·4807	3·4760	2·1118	11432	4·4596	·0238	1·6232493
43	135·0	1452·20	1849	6·5574	3·5034	2·1217	12124	4·5018	·0233	1·6334685
44	138·2	1520·53	1936	6·6332	3·5303	2·1315	12842	4·5434	·0227	1·6434527
45	141·3	1590·43	2025	6·7082	3·5569	2·1411	13584	4·5844	·0222	1·6532125
46	144·5	1661·90	2116	6 7823	3·5830	2·1506	14351	4·6249	·0217	1·6627578
47	147·6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	15144	[illegible]	·0213	1·6720979

51	160·2	2042·	2601	7·1414	3·7084	2·1954	18574	4·8198	·01961	1·7075702
52	163·3	2123·	2704	7·2111	3·7325	2·2039	19499	4·8574	·01924	1·7160033
53	166·5	2206·	2809	7·2801	3·7563	2·2124	20449	4·8945	·01887	1·7242759
54	169·6	2290·	2916	7·3484	3·7798	2·2206	21428	4·9313	·01852	1·7323938
55	172·7	2375·	3025	7·4162	3·8030	2·2288	22435	4·9676	·01818	1·7403627
56	175·9	2463·	3136	7·4833	3·8259	2·2369	23468	5·0035	·01786	1·7481880
57	179·0	2551·	3249	7·5498	3·8485	2·2448	24529	5·0391	·01754	1·7558749
58	182·2	2642·	3364	7·6158	3·8709	2·2526	25619	5·0742	·01724	1·7634280
59	185·3	2733·	3481	7·6811	3·8930	2·2603	26638	5·1091	·01695	1·7708520
60	188·4	2827·	3600	7·7460	3·9149	2·2679	27885	5·1435	·01667	1·7781513
61	191·6	2922·	3721	7·8102	3·9365	2·2754	29061	5·1776	·01639	1·7853298
62	194·7	3019·	3844	7·8740	3·9579	2·2829	30267	5·2114	·01613	1·7923917
63	197·9	3117·	3969	7·9372	3·9791	2·2902	31503	5·2449	·01587	1·7993405
64	201·0	3216·	4096	8·	4	2·2974	32768	5·2780	·01563	1·8061800
65	204·2	3318·	4225	8·0623	4·0207	2·3045	34063	5·3109	·01538	1·8129134
66	207·3	3421·	4356	8·1240	4·0412	2·3116	35388	5·3434	·01515	1·8195439
67	210·4	3525·	4489	8·1854	4·0615	2·3185	36744	5·3756	·01493	1·8260748
68	213·6	3631·	4624	8·2462	4·0817	2·3254	38130	5·4076	·01471	1·8325089
69	216·7	3739·	4761	8·3066	4·1016	2·3322	39547	5·4393	·01449	1·8388491
70	219·9	3848·	4900	8·3666	4·1213	2·3389	40996	5·4707	·01429	1·8450980
71	223·0	3959·	5041	8·4261	4·1408	2·3456	42476	5·5018	·01408	1·8512583
72	226·1	4071·	5184	8·4853	4·1602	2·3522	43987	5·5326	·01389	1·8573325
73	229·3	4185·	5329	8·5440	4·1793	2·3588	45531	5·5639	·01370	1·8633229
74	232·4	4300·	5476	8·6023	4·1983	2·3651	47106	5·5936	·01351	1·8692317
75	235·6	4417·	5625	8·6603	4·2172	2·3714	48714	5·6237	·01333	1·8750613

LE XII.—Part 7—*continued.* Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{a^5}$	$\sqrt[5]{a^2}$	Reciprocal.	Logarithm.
76	238·7	4536·	5776	8·7178	4·2358	2·3777	50354	5·6536	·01316	1·8808136
77	241·9	4656·	5929	8·7750	4·2543	2·3840	52026	5·6832	·01299	1·8864907
78	245·0	4778·	6084	8·8318	4·2727	2·3901	53732	5·7127	·01282	1·8920946
79	248·1	4901·	6241	8·8882	4·2908	2·3962	55471	5·7418	·01266	1·8976271
80	251·3	5026·	6400	8·9443	4·3089	2·4022	57243	5·7708	·01250	1·9030900
81	254·4	5153·	6561	9·	4·3267	2·4082	59049	5·7995	·01235	1·9084850
82	257·6	5281	6724	9·0554	4·3445	2·4141	60888	5·8281	·01220	1·9138139
83	260·7	5410	6889	9·1104	4·3621	2·4200	62762	5·8564	·01205	1·9190781
84	263·8	5541·	7056	9·1652	4·3795	2·4258	64669	5·8845	·01190	1·9242793
85	267·0	5674·	7225	9·2195	4·3968	2·4315	66611	5·9125	·01170	1·9294189
86	270·1	5808·	7396	9·2736	4·4140	2·4372	68589	5·9402	·01163	1·9344985
87	273·3	5944·	7569	9·3273	4·4310	2·4429	70559	5·9677	·01149	1·9395193
88	276·4	6082·	7744	9·3808	4·4480	2·4485	72645	5·9951	·01136	1·9444827
89	279·6	6221·	7921	9·4340	4·4647	2·4540	74726	6·0222	·01124	1·9493900
90	282·7	6361·	8100	9·4868	4·4814	2·4595	76843	6·0492	·01111	1·9542425
91	285·8	6503·	8281	9·5394	4·4979	2·4650	78995	6·0760	·01099	1·9590414
92	289·0	6647·	8464	9·5917	4·5144	2·4703	81183	6·1026	·01087	1·9637878
93	292·1	6792·	8649	9·6437	4·5307	2·4757	83408	6·1291	·01075	1·9684829
94	295·3	6939·	8836	9·6954	4·5468	2·4810	85668	6·1553	·01064	1·9731279
95	298·4	7088·	9025	9·7468	4·5629	2·4863	87964	6·1814	·01053	1·9777236
96	301·5	7238·	9216	9·7980	4·5789	2·4915	90298	6·2074	·01042	1·9822712
97	304·7	7389·	9409	9·8489	4·5947	2·4966	92668	6·2332	·01031	1·9867717
98	307·8	7542·	9604	9·8995	4·6104	2·5018	95075	6·2588	·01020	1·9912261
99	311·0	7697·	9801	9·9499	4·6261	2·5068	97519	6·2843	·01010	1·9956352
100	314·1	7854·	10000	1·0	4·6416	2·5119	100000	6·3096	·01000	2·0

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APPENDIX

OF

MISCELLANEOUS TABLES AND DATA.

RETAINING WALLS.
MASONRY DAMS.
THICKNESS AND WEIGHT OF PIPES.
DUTY OF HYDRAULIC MACHINES.
INDIAN HYDRAULIC CONTRIVANCES.
CONSTANTS OF LABOUR AND CARTAGE.

MISCELLANEOUS TABLES AND DATA.

Formulæ and Data for Retaining Walls.

Extracted from various articles by J. H. E. Hart, Esq., C.E.

General equation for breadth of base, $x = \sqrt{\left\{\frac{n\,\mathrm{H}}{3\,w\,(q \pm q^1)}\right\}}$

ere H = total horizontal pressure against the back of the wall.

n = the ratio of its sectional area to that of a rectangle of equal height and breadth.

w = the weight of a cubic foot of the wall.

qx = the horizontal deviation of the centre of resistance of the base from the middle of the base.

q^1x = the horizontal deviation of the centre of gravity of the profile from the middle of the base.

For vertical rectangular sections, $n = 1, q^1 = 0, x = \sqrt{\left(\frac{\mathrm{H}}{3wq}\right)}$

For plumb-faced trapezoidal sections of a top thickness (t)

$$n = \frac{x + t}{2\,x} \text{ and } q^1 = \left(\frac{t - x}{6}\right)\left(\frac{x + 2\,t}{x\,(x + t)}\right)$$

$$x = \sqrt{\left\{\frac{2\,\mathrm{H} - w\,t^2}{3\,w\,(q - \frac{1}{6})} + \left(\frac{t}{2}\right)^2\right\}} - \frac{t}{2}$$

For plumb-backed trapezoidal sections of a top thickness (t)

$$n = \frac{x + t}{2\,x} \text{ and } q^1 = \left(\frac{x - t}{6}\right)\left(\frac{x + 2\,t}{x\,(x + t)}\right)$$

$$x = \sqrt{\left\{\frac{2\,\mathrm{H} + w\,t^2}{3\,w\,(q + \frac{1}{6})} + \left(\frac{t}{2}\right)^2\right\}} - \frac{t}{2}$$

The limiting value of q to avoid the existence of tension in the sonry is $\frac{1}{6}$, but its limiting value in actual practice is $\frac{1}{8}$. In ecial cases, since it must not be so great as to cause the maxi-um pressure (P) to exceed the safe resistance (C) to crushing of e material, its values correspond as follows to the values of $\frac{\mathrm{P}}{p}$, where = the mean pressure per unit of surface of base, = sum of the rtical forces ÷ area of the base; and P is less than C.

$$q = \tfrac{1}{12},\ \tfrac{1}{11},\ \tfrac{1}{10},\ \tfrac{1}{9},\ \tfrac{1}{8},\ \tfrac{1}{7},\ \tfrac{1}{6};\ \tfrac{2}{11},\ \tfrac{1}{5},\ \tfrac{2}{9},\ \tfrac{1}{4}$$

$$\frac{\mathrm{P}}{p} = \tfrac{3}{2},\ \tfrac{17}{11},\ \tfrac{8}{5},\ \tfrac{5}{3},\ \tfrac{7}{4},\ \tfrac{13}{7},\ 2;\ \tfrac{15}{7},\ \tfrac{20}{9},\ \tfrac{12}{5},\ \tfrac{8}{3}$$

x = thickness of a vertical rectangular wall to sustain a horizontal-topped bank,

x_1 = do. for an indefinite surcharge,

x_2 = do. for a surcharge of a height c,

$x_2 = \frac{h\,x + 2\,c\,x_1}{h + 2\,c}$ where h = height of the wall.

MISCELLANEOUS DATA—*continued.*

Co-efficients for Earth Pressure against one foot in length of vertical-backed Walls for various angles of repose of earth.

For angles of repose of	27°	30°	33°	36°	39°	42°	45°
	Co-efficients of earth pressure.						
Earth horizontal at the level of the top ...	·188	·167	·147	·130	·114	·099	·085
Indefinite Surcharge to angle of repose ...	·397	·375	·351	·327	·302	·276	·250

Horizontal pressure H = co-efficient × weight of 1 cubic foot × h^2.

For walls having sloping backs the horizontal pressure is co veniently determined by Neville's well-known geometrical metho which gives the position of the plane of maximum pressure, a hence also the values of ε the inclination of that plane with the ang of repose, and A the sectional area of effective pressure, in the gene expression for horizontal thrust, H = A tan ε × weight of 1 cu foot of the earth.

For water pressure H = 31·2 × h^2.

***Working Loads** or safe units of pressure adopted in existing structures.*

(From Spon's "Dictionary of Engineering.")

	Tons on the square foot.
Soft rock foundations	9
Concrete	3
Earth	1½
Ashlar masonry, limestone, Britannia Bridge	16
" " granite, Saltash Bridge	10
" backed with rubble, Peniston Viaduct	6
Rubble masonry, sandstone in Aberthaw lime, Pont y Pridd	20¼
" " limestone in chalk lime, Barentine Viaduct	3¼
" " in hydraulic lime, Almanza Dam ...	12·3
" " " " Ban	7·3
" " " " Furens	6·
" " " " Tulsi	8·9 to 6·9
Brickwork, London paviors' in cement, Charing Cross Bridge	12
" Staffordshire blue brick in cement, Clifton Suspension Bridge	10
" red Birmingham in lias lime, Railway Viaduct	7
Cement mortar	20 to 33
Lime mortar	2½ to 5¼

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MISCELLANEOUS DATA—*continued.*

Table of Weights of Materials

(From Spon's "Dictionary of Engineering.")

	Angle of repose.	Specific gravity.	Weight of a cubic foot.
y	30° to 40°	1·95	120
et	15 to 20	2·17	135
ommon dry	46	1·64	102
lay and sand	54	1·5 to 1·7	97 to 106
	37	1·5 to 1·9	96 to 120
garden	35 to 45	1·4	70 to 90
ry fine	34 to 40	1·4 to 1·6	84 to 97
amp	34 to 40	1·9	118
, loose	39	2·2	139
and traps		3 to 2·4	187 to 155
red		2·16	135
ommon		1·76	110
tock (London)		1·84	115
rk in cement		1·92	120
in new mortar		1·87	117
in old mortar		1·52	95
new		1·61	100
sonry		2·34	148
		3·05 to 2·25	190 to 141
masonry		2·75	172
nes		2·54 to 1·86	159 to 116
, new		1·9	119
old		1·42	89
nes		2·67 to 1·38	168 to 88
		2·9 to 2·5	180 to 157

weight = $\frac{7}{8}$ that of stone + $\frac{1}{8}$ that of mortar.
weight = $\frac{3}{4}$ to $\frac{2}{3}$ that of stone + $\frac{1}{4}$ to $\frac{1}{3}$ that of mortar.

afe working load for masonry and brickwork is that for the ised; but in ordinary calculation, 5 tons per square foot for rk and rubble, and 30 for ashlar in cement, is generally allowed.

MISCELLANEOUS DATA—*continued.*

Dimensions of Trapezoidal Masonry Dams, having both faces battering, for heights up to 40 feet. (By the Author.)

	Good rubble.	Inferior rubble.	Brickwork.
Height of dam ...	[illegible]	H	H
Thickness at top ...	[illegible]	·2H	·3H
Thickness at bottom	[illegible]	·6H	·7H
Front batter ...	[illegible]	1 in 15	1 in 15
Back batter ...	[illegible]	1 in 3	1 in 3
Sectional area ..	[illegible]	$·4H^2$	$·5H^2$

Dimensions of Trapezoidal Masonry Dams, having the water face vertic for heights up to 40 feet.

Weight of masonry per cubic foot	140 lbs.	120 lbs.	100 lbs.
Height of dam	H	H	H
Thickness at top	·24H	·25H	·28H
Thickness at bottom ...	·48H	·51H	·56H
Water face	Vertical	Vertical	Vertical
Outer face...	1 in 4·25	1 in 4	1 in 3·57
Sectional area	$·36H^2$	$·375H^2$	$·42H^2$
Weight per unit of length	$50H^2$	$45H^2$	$42H^2$
Mean pressure	104H	90H	75H
Maximum pressure ...	416H	360H	300H

These data apply to the same limiting value of q, the ratio to th breadth of the base of the distance along it from the foot at whic the direction of the resultant pressure cuts, which is taken at one third. A slight modification of the above section may be used fo heights up to 50 feet. For lofty dams, the process and rules Rankine for obtaining the dimensions of dams with curved profil under different conditions yield correct results by means of short an simple calculations.

[redacted]

v

MISCELLANEOUS DATA—*continued.*

cknesses, Sizes, and Weights of Cast Iron Pipes. (Box.)

Safe Thickness for Various Pressures.

r	Head of water in feet.					
	For Gas.	100	250	500	750	1000
	Thickness in inches.					
	·27	·28	·29	·30	·31	·33
	·29	·3	·31	·33	·35	·37
	·3	·31	·33	·35	·37	·41
	·32	·33	·35	·38	·41	·44
	·35	·37	·39	·43	·47	·51
	·37	·39	·42	·47	·52	·57
	·39	·42	·45	·51	·57	·63
	·41	·44	·48	·55	·62	·69
	·43	·46	·51	·59	·67	·75
	·45	·48	·53	·63	·72	·81
	·46	·50	·56	·66	·76	·86
	·49	·54	·61	·73	·85	·97
	·53	·59	·68	·83	·98	1·13
	·57	·64	·75	·93	1·11	1·29
	·6	·69	·81	1·02	1·23	1·44
	·64	·73	·88	1·12	1·36	1·60
	·69	·81	1·00	1·29	1·59	1·89
	·75	·89	1·11	1·47	1·83	2·19

zes and Weights of Socket Pipes for a Head of 200 Feet.

r.	Length without socket.	Depth of socket.	Lead joint, thick,	deep,	weight.	Average weight of pipe.		Average weight of quarter bends.	
	feet	inches	"	"	lbs.	cwt.	lbs.	cwt.	lbs.
	6	3	¼	1¼	1·2		42		28
	6	3	¼	1½	1·4		56		30
	6	3½	¼	1½	1·6		67		37
	9	3½	5/16	1¾	2·3	1	8		45
	9	4	5/16	2	4·	1	56		75
	9	4·	5/16	2	5·	2			84
	9	4½	5/16	2¼	6·5	2	56		87
	9	4½	5/16	2¼	7·7	3	12	1	25
	9	4½	5/16	2¼	8·2	3	80	1	58
	9	4½	5/16	2¼	10·4	4	28	1	74
	9	4½	5/16	2½	11·5	4	98	3	84
	9	4½	5/16	2¾	15·	6	56	3	105

MISCELLANEOUS TABLES—*continued.*

Hydraulic Machines:—Return of Motive Power.

Deduced from Morin's Experiments.

	Proportion of Motive Power yielded.
Lift pump	·316
Force pump ...	·516
Fire engine ...	·233
Chinese wheel	·36 ·59
Flash wheel ...	·75
Wirtz pump	·181 ·640
ROTARY.	
Stotz pump ...	·43
Leclerc	·307
CENTRIFUGAL.	
Piatti	·20
Appold	·70
Gwynne ...	·190 ·300
Girard	·300
Vertical helix ...	·19
WATER RAMS.	
Montgolfier ...	·47 ·80
Caligny	·43
Foex	·55
Dartige's balance ...	·72
Belidor	not used
Huelgoat	·45
Pfetsch	·771

	Proportion of Motive Power yielded.	Proportion of
FIRE ENGINES.		
weather	·572	·9
... ...	·625	·68
n	·452	·91
... ...	·300	·91
... ...	·194	·9
... ...	·210	·9
DRAINAGE PUMPS.		
ot	·690	·9
ch	·600	·9
n	·519	·9
... ...	·502	
SUPPLY PUMPS.		
At Ivry (feeder alone) ..	·230	
At Ivry (three pumps)...	·530	
At St. Ouen	·696	
At Lisbon (Farcot) ...	·652	
Solid piston pumps ...	·900	
OSCILLATING PUMPS.		
Vascile's fire-engine ...	·50	
Gray's oscillating ...	·45	

(From Neville.)

Overshot wheels ...	·76	Barker's mill ...	·16 ·35
Breast wheels ...	·52	Ballysillan vortex ...	·75
Very wide breast-wheels	·70	Tremont vortex	·794
Undershot wheels...	·33	Montgolfier's ram ...	·65
Floating turbine ...	·38	Easton and Amos ram ..	·67
Impact turbine	·16 ·40	Water-pressure engines...	83

Duty of Indian Hydraulic Contrivances.

MACHINE.	DETAILS.				WORK DONE.			Power exerted.	Co-efficient of reduction.	Power utilized.
	Lift in feet.	Men.	Oxen.	No. of lifts per hour.	Cubic feet raised.		Gallons per day of 8 hrs.			
					per lift.	per hour.		foot-pounds per second.		foot-pounds per second
Southern India.										
1. Picotah	10	2	...	600	·83	498	24,900	80	·80	64
2. Picotah	20·5	2	...	300	1·44	482	21,600	141	·80	112
3. Picotah	20·5	2	...	240	1·60	384	19,200	127	·80	101
4. Dal	10	6	...	1320	·33	443	22,150	71	·70	50
5. Mot	11	...	2	44	1·77	78	3,900	14	·70	10
6. Mot	20	...	2	90	2·00	180	9,000	58	·70	40
7. Mot	45	...	2	32	1·75	56	2,800	40	·70	28
8. Double mot	22·5	...	2	180	1·32	238	11,900	87	·60	52
9. Common pump	10	1	...	60	per minute. 7·77	466	23,300	74	·50	37
Berar.										
10. Mot	16	1	2	35	per lift. 5·81	203	10,150	52	·70	36
Northern India.										
11. Baling	5	2	...	1200	·33	400	20,000	32	·75	24
12. Beam and bucket	16	1	...	180	·45	82	4,100	20	·80	16
13. Windlass	4	1	...	84	2·00	168	8,400	11	·50	5
14. Dal	5	2	...	84	5·00	420	21,000	33	·70	23
15. Single mot	40	1	2	70	3·00	560	28,000	132	·70	92
16. Double mot	40	1	2	85	3·00	255	12,750	164	·60	98
17. Single chain of pots	40	1	2	168	·75	126	6,300	160	·55	88
18. Double chain of pots	40	1	2	138	2·40	330	16,500	212	·60	127
19. Common pump	25	4	...	2400	·10	240	12,200	97	·50	48
20. Lift and force pump	40	4	...	1440	·20	288	14,400	184	·60	110

MISCELLANEOUS DATA—*continued.*

Constants of Labour. (Hurst.)

EARTHWORK.

Excavator's Work per cubic yard, in terms of a day's labor of 10 h
for different descriptions of soil.

Days of a Laborer.	Materials.	
	Soft. Days.	Moderate. Days.
Excavating only	·050	·100
„ in rock requi g ...		
	Light.	Heavy. We
Throwing 5 feet high, or ks ...	·048	·055
Filling barrows	·045	·052
Removing with wheelbar. yards' distance	·026	·030
Filling at backs of walls ...	·048	·055
Ramming earth in 6-inch ...	·040	
„ „ 12-inch	·025	
Levelling earth from barrow-heaps without throwing	·012	·019

Levelling and trimming slopes per s. yard	·020	to	·030
Turf 4 inches thick, cutting and stacking only, per s. yard	·045		
„ „ resodding only, per s. yard ...	·065		

Days of driver, horse, and cart.

Removing 220 yards distance, per c. yard	·035	to	·040
Each additional 220 yards „ „	·020	to	·025

N.B.—The vertical transport of earth is equal to 15 times the horizontal distance when barrows are used, and 12 times when h and carts are employed.

Days of an Indian Coolie.

	Sand.	Gravel.	Sto
xcavating down to 9 feet, carrying to 25 yds. in a basket and depositing up to 6 ft. ...	1·25	2.00	
Excavating down to 15 feet „ „	2·00	2·75	
Add for each 3 feet more of depth or height of delivery, or for each 15 yards' additional distance...	·25	·25	

[redacted]

MISCELLANEOUS DATA—*continued.*

Constants of Labour. (Hurst.)

BRICKLAYERS' WORK.

me in days of 10 hours in which work can be performed.

One Bricklayer's Laborer.		Days.
ncrete, wheeling and throwing from a stage,	1 c. yd. in	·300
ortar with a shovel	1 c. yd. in	·720
rse pug-mill mixes 25 cubic yards of mortar ...	in	1·
p and stacking bricks without moving, per 1,000	...	·150
„ if handed to him	„	·100
bricks for facings	...	·300
down old brickwork in mortar, cleaning and g	1 c. yd. in	·410

One Bricklayer and Laborer.		Days.
k in mortar to walls, exclusive of face work,	1 c. yd. in	·320
in cement	„	·373
in mortar to covering arches	„	·410
flat joint in mortar and raking out mortar	1 s. yd. in	·110
flat joint in cement and raking out cement	1 s. yd. in	·170
uck in cement and raking out cement joints	„	·258
ith stock bricks on edge in mortar ...	„	·086
„ „ in cement ...	„	·100
d jointing in cement 3-inch drain pipes	1 l. yd. in	·024
„ „ 6 „	„	·048
„ „ 9 „	„	·069
„ „ 12 „	„	·093
„ „ 18 „	„	·150

One Bricklayer only.		Days.
each fair face to brickwork and pointing	per s. yd.	·080
each fair face in malms or facing of superior	per s. yd.	·117
each fair face in malms, circular to tem-	per s. yd.	·189
tting to brickwork	„	·135
„ „	„	·360

CELLANEOUS DATA—*continued.*

Constants of Labour—(Hurst.)

Masons' Work.

Days of a Laborer.		Days.
Rubble Stone.—Filling barrows	per cubic yard	·060
„ Removing 25 yards, and returning	„ „	·040
„ Unloading barrows	„ „	·080
„ Taking down o[illegible] mortar, cleaning and [illegible] ...	„ „	·600
Breaking stone to 1½" [illegible] ...	„ „	·700
Do. do. granite [illegible]s ...	„ „	·930
Spreading the same for m[illegible] ...	per square yard	·022

Days o[illegible] Laborer.		Days.
Rubble masonry, dry in fo[illegible] ...	per cubic yard	·240
„ „ in mort[illegible]tions...	„ „	·310
„ „ all beds [illegible]d ...	„ „	·460
„ „ in cement do.	„ „	·570
Ashlar masonry, 12" thick and in 12" courses, rubble with chisel-drafted margins	„ „	2·160
Cubed stone hoisted and set in mortar	„ „	·765
„ „ „ in cement	„ „	·945

Days of a Mason only.		Days.
Add to rubble masonry for each fair face	... per square yard	·090
„ „ if hammer dressed ...	„ „	·360
„ „ if curved	„ „	·414
Squaring 2" flags for paving	„ „	·072
„ 4" „	„ „	·135

Days of a Mason on various sorts of stone.

		Caen. Days.	Portland. Days.	Granite. Days.
Whole sawing, or axing,	per square yard	·270	·540	1·270
Plain work	„ „	·540	·765	1·800
„ circular	„ „	·900	1·395	2·160
Sunk work	„ „	·675	1·080	2·135
„ circular	„ „	1·035	1·575	2·925
Moulded work	„ „	1·395	1·800	3·825
„ circular	„ „	1·800	2·700	4·905

Constants of Labor per ton and per 100 cubic feet in terms of a day's work of a cart.

Distance of "lead" in miles.	No. of trips.	Cost of one trip.	Constants for one ton.				Constants for 100 cubic feet.								
			For a weight of load of				For a capacity of load in cubic feet of								
			8 cwt.	8½ cwt.	9 cwt.	10 cwt.	6	7	8	9	10	11	12	[illegible]	16
3/16 to ¼	16	·0625	·156	·149	·139	·125	1·042	·893	·781	·694	·625	·568	·521	·417	·391
¼ ,, ⅜	12	·083	·208	·196	·185	·167	1·389	1·190	1·042	·926	·833	·757	·694	·556	·521
⅜ ,, ½	10	·1	·25	·235	·222	·2	1·667	1·429	1·25	1·111	1·	·909	·833	·667	·625
½ ,, ¾	8	·125	·313	·294	·278	·25	2·083	1·786	1·563	1·389	1·25	1·136	1·042	·833	·781
¾ ,, 1	6	·167	·417	·392	·370	·333	2·778	2 381	2 083	1·852	1·667	1·515	1·389	1·111	1·042
1 ,, 1¼	5	·2	·5	·471	·445	·4	3·333	2·857	2·5	2·222	2·	1·818	1·667	1·333	1·25
1¼ ,, 1¾	4	·25	·625	·588	·556	·5	4·167	3·571	3·125	2·778	2·5	2·273	2·083	1·667	1·563
1¾ ,, 2½	3	·333	·833	·784	·741	·667	5·556	4·762	4·167	3·704	3·333	2·030	2·778	2·222	2·083
2½ ,, 3½	2	·5	1·25	1·176	1·111	1·0	8·333	7·143	6·250	5·556	5·	4·545	4·167	3·333	3·125
3½ ,, 4½	1½	·667	1·67	1·57	1·48	1·33	11·111	9·524	8·333	7·407	6·667	6·061	5·556	4·444	4·167
4½ ,, 5½	1¼	·8	2·0	1·88	1·78	1·6	13·333	11·429	10·	8·889	8·	7·273	6·667	5·333	5·
5½ ,, 8	1	·1	2·5	2·35	2·22	2·0	16 667	14·286	12·5	11·111	10·	9·091	8·333	6·667	6·25

MISCELLANEOUS DATA—continued.

Indian Coinage, Weights, and Measures.

The reductions from Indian data in the statistics are based on the assumption that the Rupi is equivalent to two shillings, and the Man or Maund to 80 lbs. avoirdupois. To aid the reader in any reductions from casual Indian data he may wish to make, the following equivalents may be useful.

The Rupee is the basis of British-Indian coinage and weights, and its weight is called a Tola.

			£.	s.	d.		Grains Troy.
1 Pie		=			⅛	and weighs	33⅓
1 Anna	= 12 Pie	=			1½	and weighs	400
1 Rupee	= 16 Annas	=		2	0	and weighs	180
1 Mohar	= 16 Rupees	=	1	12	0	and weighs	180

The established British-Indian weights are :—

1 Tola = = ·41143 oz. Avoir.
5 Tolas = 1 Chittak = 2·05714 oz. Avoir.
16 Chittaks = 1 Seer or Ser = 2·05714 lb. Avoir.
40 Seers = 1 Man or Maund = 82·2857 lb. Avoir.

The Seer is nearly a Kilogramme.

1 lb. Troy weighs 32 Tolas, and 1 lb. Avoirdupois 38·89 Tolas.

There are no measures of capacity, liquid and dry goods being estimated by weight.

The measures of length are the English yard or gaz, and the English mile, which has now superseded the very variable kos.

The measure of surface, the bigha, is not yet generally superseded by the English acre, its value in different places is :—

In Bengal ...	1600 s. yards.	In Northern India	3025 s. yards.
At Banaras and Ghazipur	3136 s. yards.	In Orissa ...	4840 s. yards.
The Madras Kani	6400 s. yards.	In Tirhut ...	4225 s. yards.
At Bombay ...	3406 s. yards.	The English acre .	4840 s. yards.

The local weights, the seer, man, and kandi, vary everywhere in Southern India; in the towns of Madras and Bombay they are thus :—

Madras.	Bombay.
1 Seer = ¼ viss = 10 oz. Av.	1 Seer = 11·2 oz. Av.
40 Seers = 1 Man = 25 lb. Av.	40 Seers = 1 Man = 28 lb. Av.
20 Mans = 1 Kandi = 500 lb. Av.	20 Mans = 1 Kandi = 560 lb. Av.

The other local weights and measures are both voluminous and doubtful, varying in almost every district.

[redacted]

DRAULIC MANUAL.

PART II.

CONSISTING OF

HYDRAULIC STATISTICS

AND

DIAN METEOROLOGICAL STATISTICS,

FOR THE USE OF ENGINEERS.

COLLECTED AND REDUCED

BY

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LONDON:
. H. ALLEN & CO., 13, WATERLOO PLACE, S.W.
1875.

HYDRAULIC STATISTICS.

GRAVITY AND TEMPERATURE.

STATISTICS OF RIVERS.

FALLS AND CURVES OF INDIAN RIVERS.

DISCHARGES OF INDIAN RIVERS.

BRIEF ACCOUNTS OF INDIAN RIVERS.

FINANCIAL STATISTICS OF INDIAN CANALS.

CANAL STATISTICS.

BRIEF ACCOUNTS OF INDIAN CANALS.

DATA OF ENGLISH RESERVOIRS.

SPANISH RESERVOIRS AND DAMS.

FINANCIAL STATISTICS OF INDIAN RESERVOIRS AND TANKS.

BRIEF ACCOUNTS OF THE SAME.

WATERWORKS OF INDIAN CITIES.

IRRIGATED CROPS AND PLANTATIONS.

WATERINGS AND WATER-RATES.

DESCRIPTIONS AND ANALYSES OF WATER AND SILT.

[redacted]

ynamic Force of Gravity at the Sea Level, and the Mean Temperature, for various Latitudes.

	Gravity.	Latitude.			Temperature.
		°	′	″	
gen	32·2526	79	49	58	...
nd	32·2435	74	32	19	...
... ...	...	65	30	0	34·38
hetlands) ...	32·2173	60	45	25	...
... ...	...	60	27	0	40·28
... ...	32·2040	55	58	41	...
gh	32·2040	55	57	0	45·64
... ...	32·1908	51	31	8	50·74
... ...	32·1895	51	2	10	...
... ...	32·1820	48	50	14	53·65
x	32·1691	44	50	26	57·82
ier	...	43	36	0	59·03
... ...	32·1668	43	7	9	...
rk	32·1600	40	42	43	...
i	32·1380	35	0	0	...
Good Hope ...	32·1403	33	55	15	...
... .	32·1412	33	51	39	61·3
aneiro	32·1121	22	55	13	...
nagar	...	22	52	0	75·10
s	32·1147	20	9	19	...
... ...	...	18	53	0	80·60
... ...	32·1050	17	56	7	...
... .	32·0917	10	38	56	...
eone	32·0927	8	29	28	...
n	32·0959	7	55	48	...
... ...	...	6	58	0	80·90
as Isle	32·0930	0	24	41	...
sle	...	0	1	34	...
... ...	...		...		81·50
ised in the work- les	32·2				

Statistics (mostly from Beardmore) for a few Rivers in various parts of the World, given for comparison with Indian Rivers.

	N	Q_3	Q_1	q_2	q_1	n_2	Proportion to down-fall. (1.)
	Catchment Area.	Mean annual discharge.	Maximum or minimum discharge.	Mean discharge per square mile.	Maximum or minimum discharge per square mile.	Depth run off annually.	
	Sq. miles.	C. ft. per sec.	C. ft. per sec.	C. ft. per sec.	C. ft. per sec.	Feet.	
Mississipi … … … …	886 000	550 000	1 200 000	·62	1·13	0·70	·24
Ganges at Benares … … …	180 000	250 000	1 285 000	1·38	7·15	1·57	
„ at Kot … … … …	192 000	…	(mi[illegible]	…	(min.) ·071	…	
„ at Sikrigali … … …	330 000	500 000	1[illegible]	1·51	5·45	1·71	
Nile at Cairo … … … …	600 000	167 000	[illegible]	[illegible]	[illegible]·60	0·32	
Kaveri … … … … …	32 000	17 000	[illegible]	[illegible]	[illegible]·85	0·60	·75
Po at Pontelagoscuro … … …	26 754	61 000	221 000	2·27	8·15	2·57	
Ticino below L. Maggiore … …	2 420	11 000	63 000	4·45	8·15	5·03	
Adda below L. Como … … …	1 670	7 000	28 000	3·95	16·91	4·47	
Rhine at Lanterburg … … …	63 000	39 000	177 000	·63	2·49	0·71	
Rhone at Avignon … … …	35 745	61 000	353 000	1·69	9·71	1·91	·75
Garonne at Armande … … …	20 028	24 000	371 000	1·19	18·51	1·35	·66
Seine at Paris … … … …	17 111	8 000	63 000	·51	3·71	·57	·36
Thames at Staines … … …	3 086	1 600	6 600	·54	2·16	·61	
Medway at Preston … … …	481	…	2 300	…	4·85	…	
Shannon at Killaloe … … …	4 571	…	73 000	…	16·00	…	

N.B.—$n_2 = 1·15 \times q_2$ in the above columns, [illegible]

	Length.	Drainage area.	Ordinary discharge.	Tidal portion.	Surface slopes on the lower portions only.	
	Miles.	Sq. miles.	C. ft. per sec.	Miles.	Feet per mile.	
Mississipi and Missouri ...	4400	1 226 600	665 900	...	O. ·233 F. ·271	From Ohio to the sea.
Amazon	4000	...		400	·195	
Nile	2240	520 200	28 100	...	L. ·271 H. ·458	Cairo to the sea.
Ganges	1680	432 480	207 000	...	·281	Rajmahal to Mirzapur.
Irrawadi	...	...	75 000	105	L. ·133 H. ·317	
Rhine	700	88 853	66 000	...	·642	Cologne to the sea.
Rhone	560	38 329		...	2·015	Besançon to the sea.
Thames	204	5 000	1 700	66	·076	Teddington to Yantlet Creek.
Severn	180 Nav	8 580	...	68	·583	Inward point to Austhead.
Clyde	98	945	800	22	·108	Broomielaw to the sea.
Boyne	60	700	3 000	2		

N.B.—O. means ordinary; F. flood; L. low-water; H. high-water.

[redacted]

s of the Breadths, &c. of various large Rivers at their Entrances (from Heywood).

	Ganges.	Kistna.	Godaveri.	Kaveri.	Mahanaddi.	Mississipi.
Extreme breadth	5½ miles	1½ miles	4 miles	1 mile	3 miles	...
Extreme breadth of channel	2½ "	1¾ "	2¼ "	¾ "	...	4000 feet
Least breadth of channel	1½ "	⅝ "	1⅛ "	⅛ "	¼ mile	2750 "
Fall per mile, in feet	...				1·67	0·27
Rise in monsun, in feet	27				32	61
Greatest depth in dry season	30	10	10	6	...	75
Surface current in floods	4 to 7 miles	7½ miles	4½ miles	6 miles	...	5 miles
Flood section, in square feet	288 000	153 000	216 000	37 800	...	240 000
Flood discharge, in cubic feet per second ...	1 800 000	1 500 000	1 500 000	300 000	1 800 000	1 500 000
Least discharge in dry season	45 000	1 125	2 250	None	...	...
Longest duration of flood	40 days	10 days	10 days	10 days	12 hours	80 days

[redacted]

The Areas of the River Basins of India.

Western Basins.

	Square miles.
	372 700
Desert	68 700
	22 400
var and Kach pe-nsulas	27 600
	15 500
ada	36 400
	27 000
ern Ghats and Coast	
ies	41 700
rmatti	9 500
ern Banas	6 300
dur	1 800
Total	629 600

Burmese Basins.

di	158 000
ng	18 300
en	62 700
an Basins	29 700
serim Coast	14 200
Total	375 700
maputra	361 200

Eastern Basins.

	Square miles.
Ganges	391 100
Sabanrekha	11 300
Baitarani	11 900
Brahmani	15 400
Mahanaddi	43 800
Godavari	112 200
Orissa Coast	22 200
Coromandel Coast	10 300
Lake Pulicat	6 700
Lake Koler	3 100
Kistna	94 500
Pennar	20 500
Palar	6 300
South Pennar	6 200
Vellar	4 500
Kaveri	27 700
Vaiga	9 800
Tambaravari	3 600
Vaipar	3 900
Total	705 000

Lateral curves of rivers of fixed regimen. (Fergusson.)

River Lengths.	Direct distance.	Distance by river.	Width of stream, low water, dry season.	Oscillations.	Length of oscillation.
	Miles.	Miles.	Feet.	Number.	Miles.
Ganges.					
Allahabad to Chunar	62	104	3500	17	3·7
Chunar to Baxar	80	113	4000	20	4
Baxar to Patna	74	96	5000	15	5
Patna to Manghir	82	106	6000	11½	7
Manghir to Rajmahal	96	108	7000	10	9·5
Rajmahal to Rajapor	90	100	7000	10	9
Rajapor to Patna	30	44	4000	6	5
Patna to Jafirganj	32	36	3000	8	4
Bhagaratti.					
Choka to Naddia	96	120	1900	62	1·5
Naddia to Chogdah	24	30	2000	9	2·5
Chogdah to Calcutta	34	42	3000	11	3
Jellinghi.					
Jellinghi to Naddia	50	112	1000	48	1·9
Mata bagh.					
Ganges to Cumar	18	28	1500	9	2·0
Cumar to Kissanganj	30	50	600	46	·65
Kissanganj to Chogdah	24	29	600	47	·6

[redacted]

Fall in feet per mile of Indian Rivers.

NORTHERN INDIA.

he Ganges.	
at Sukertal	1·5
from Gurmaktesar to 60 miles south	1·25
hanpur to Allahabad	·75
he Bhagiratti for 190 miles; between Rajmahal and Mirzapur	·281
he Jamna, at Agra	1·25
he Indus, near Sakkar	·75
he Son, in the Punjab, at Lahor-road bridge	14·
he Markanda, at Hassanpur	2·72
he Mahanaddi,	
lower 100 miles	1·4
next 100 miles	2·2

SOUTHERN INDIA.

he Godavari,	
Sironcha to Palmilla	·5
Enchampilli to Damagudiam	1·0
Damagudiam to head of delta	·5
through delta to sea	·5
he Pranhita, Tallodhi to Sironcha, 90 miles	1·0
he Warda,	
above the Wunna	4·0
below it to Tallodhi	1·0
he Wainganga, Kampti to Tallodhi, 192 miles	2·8
he Kistna, Bezwara to sea	1·0

SOUTHERN INDIA.

The Tambrapurni, at Strivigantam	2·5 to 3·0
The Tungabaddra, Dharwar	2 to 2·5
The Warda, Dharwar	2·
The Malparba, Belgaum	1·25 to 1·5
The Gatparba, Belgaum, below Gokak	1· to 2·
The Nira, Puna, above Ramlishwar	4·6
The Indarauni, Puna	2·75
The Bhima, Puna, Sarwali to Deksal	2·75
The Siena, Sholapur, above Undogaum	2·75
The Krishna, Sattara,	
above Kursi	4·7
Kursi to Bahey	1·9
Bahey to Yerla	1·4
below Yerla	·6
The Koina,	
Helwak to Karrar	1·3
Karrar to Bahey	·4
above Bamnoli	6·0
The Yerla, Krishna to Chickli	8·8
The Mann, Diguchi to Manswar	5·5
The Kaveri,	
above the Kalerun	3·5
thence to Seringham	3·5 to 2·0
Seringham to sea	1·0
The Kalerun,	
from the Kaveri to Seringham	3·1 to 1·6
Seringham to sea	1·6 to ·6

Flood discharges of Indian Rivers, according to various reports.

	Catchment Area.	Flood Discharge.	Discharge per sq. mile.	Coefficient (c) in the [illegible]
	Sq. miles.	C. ft. per sec.	C. ft. per sec.	*
NORTHERN INDIA.				
Ganges at Rajmahal	2[illegible]00	1 350 000	4·7	1·1
Combined Mahanaddi and Katjuri in flood of 1834 ...	[illegible]00	1 850 000	27·6	4·6
Jamna at Allahabad	[illegible]00	1 333 000	11·3	2·
Son (Bengal) at causeway .	[illegible]00	1 700 000	50·0	7·
Indus at Sakkar	[illegible]00	380 000	15·2	0·3
Son (Punjab) at Lahor-road bridge	[illegible]00	96 000	26·6	2·
Markanda at Hassanpur, 1845 .	[illegible]200	47 838	39·8	2·
Sai at Rai Bareli bridge ...	960	16 500	17·2	1·
Sai at railway bridge	240	12 000	50 0	2·
Gumti at Lakhnau bridge ...	2 000	22 366	11·2	0·8
Gumti at Saltanpur bridge ...	3 600	39 000	10·8	0·8
Loni at railway bridge ...	120	4 600	38·3	1·3
Kalliani at Lakhnau bridge ...	360	17 758	49·3	2·1
Morna (Berar) at railway bridge	211	122 715	581·	20
Nalganga at railway bridge ...	213	153 846	722·	24
SOUTHERN INDIA.				
Godavari at Rajamandri ...	120 000	1 350 000	11·2	2·3
Kistna at Bezwara	110 000	1 188 000	10·8	1·9
Tumbadra at Karnul	20 000	270 000	13 5	1·6
Kaveri at Frazerpett	415	111 000	267 3	12·5
veri at Seringham	28 000	472 500	16·9	2
ner at Nellor	20 000	359 100	18·1	2
ar at Arcat	3 700	270 000	74 2	5·7
nbrapurni at Palamcotta ...	587	189 000	324·0	16
uettar at Alligyapandrapuram	486	29 700	60·8	3
Vigay at Madura	1 600	43 200	27·0	2
Manjilanthi at Balagunta ...	90	10 800	121·5	4
Gadanamathi	29	28 088	972·0	23·
Varhazanamathi at Periacolam	41	8 100	202·5	5·
Irriti (Malabar)	336	149 850	446·0	19·

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* See pages ix. and lxx. of working tables

Flood Discharges of Indian Rivers on the East-Indian Railway, by S. Power, Esq., Chief Engineer.

	Waterway.			Recorded flood level.	Estimated rainfall run off through, in inches per hour.	Discharge per square foot of waterway to carry off estimated rainfall.	Mean velocity through in order to carry this off.	Estimated addition to waterway necessary to reduce this velocity below 2·5, existing waterway being = 1.	Rainfall of district.
	Catchment area in square miles.	Lineal feet.	Square feet below recorded flood level.						
						C. ft. p. sec.	Feet p. sec.		
Kurriamnassa	3 400	4 000	39 000	11	·25	162	15	6	Comparatively light.
Son	23 000	14 200	172 500	8	·125	11	10	4	
Punpun and Hallshar valleys...	9 000	11 086	123 648	14	·25	162	11	4	
Kinal River	1 100	1 542	11 002	10	·25	162	16	6	
Hill streams west of Jamalpur	240	1 670	7 385	80	·5	323	10	4	
Do. Jamalpur to Sahibganj	2 650	6 796	59 486	23	·5	323	14	5	Heavier than that west of Manghir.
Do. Sahibgenj to Tinpahar	52	1 641	7 952	155	.5	323	2	1	
Do. Tinpahar to Baluva ...	66	1 176	2 476	144	·5	323	2	1	
Gumani River	520	1 639	10 165	20	·5	323	16	6	
Hill streams between Gumani and Mullarpur	1 200	5 824	53 301	44	·5	323	7	3	
Adjai and Mor valleys ...	3 640	7 752	120 300	30	·5	323	10	4	
Khanu to Haurah	...	14 000	65 000	...	...	...	...	...	

Discharges of Indian Rivers.

The River Son, Bengal.

Date.	Place.	
8 Jan. 1855	At causeway and headworks of canal.	5 7
1 Feb. 1855	"	4 0
1 Mar. 1855	"	11 0
Ordinary minimum	"	4 0
Extreme drought	"	9

The M[illegible]i and Katjuri.

Flood of 1855	Below [illegible] with the Beropa	1 040 0
Flood of 1855	The K[illegible]d Kokai.	780 0
	Mahanaddi Series, Total	1 820 0

The River Jamna.

6 June 1872	Mandawala	13
6 June 1872	Bud	51
29 July 1872	Chaogaon	1446
19 Dec. 1872	Railway bridge	21
19 Dec. 1872	West Ghat	20
19 Jan. 1873	Railway bridge	25
20 Jan. 1873	West Ghat	29

The River Satlaj.

Jan. 1856	Proposed site for headworks of Canal.	27
2 Feb. 1857		41
26 Jan. 1859	" "	4
20 Dec. 1859	" "	4
21 Jan. 1861	" "	4

N.B.—There is reason to believe that these are in excess.

[redacted]

Discharges of Indian Rivers—continued.

The River Ravi.

	Place.	Discharges in cubic feet per second.
1872	Shahdera, Lahor 94 miles.	703
1872	Alpah, below escape 147 miles.	879
1872	Bhátiah	509
1873	Shahdera	687
1873	Alpah	478
1873	Bhatiah	271
, 1872	Sidhuri	7 689
1872	"	13 452
1872	"	1 866
1873	"	2 296
1873	"	3 579

The River Bias.—At Naushehra.

, 1872	7 498	19 Dec. 1872.	4 901
1872	8 797	19 Jan. 1873.	5 117
1872	3 464 at Pakhowal		

The River Indus.—At Kalabagh.

.871	21 220	Jan. 1873.	20 541
.872	18 657		
1872	21 878		
.873	20 781		
.873	18 657 at Dera-Ghazi-Khan.		

The River Kuram.—At Kalabagh.

.873. 545 (included with the Indus discharges).

The River Indus, in 1872-73.—At Dera-Ghazi-Khan.

Average gauge readings for each month.

72	6·27	Aug. 1872	7·97	Dec. 1872	3·46
'2	7·32	Sept. 1872	6·19	Jan. 1873	3·55
72	9·28	Oct. 1872	4·85	Feb. 1873	3·23
'2	9·81	Nov. 1872	3·98	Mar. 1873	3·58

Rivers of Maisur.—The Kistna System.

The Kistna River System.	Feeders in Maisur.	Length in Maisur.	Area over which the drainage is unintercepted in tanks in Maisur.	Area over which the drainage is intercepted by tanks in Maisur.	Total area of catchment basin.	Percentage of whole area under tank system.	Rises	Falls into
		Miles.	Sq. miles.	Sq miles.	Sq. miles.	per cent.		
Kistna ... (Falls into Bay of Bengal in lat. 15° 45'.)	Warda	47	430	180	610	·29	Sagar, has a few minor anicuts.	River Tungabaddra, 35 miles below Harrihar; may be further utilized
	Choardi	43	None.	510	510	[illegible]	[illegible] of K. [illegible] has a [illegible]	River Tungabaddra, 10 miles above Harrihar; may be further utilized.
	Tunga	149	1287	100	1 389	[illegible]	[illegible] ° 16'. [illegible] ders.	Kudli, 10 miles NE. of Shemogah.
	Baddra	160	1500	175	1 675	10·30	Do. many anicuts on feeders.	Do. Proposed Lackawally reservoir.
	Sulikerri ...	45	281	799	1 080	77·60	S. of Chennagherri, feeds the Sulikerri tank.	River Tungabaddra at Harrihar.
	Chinna Haggri ...	53	168	356	524	67·90	(Not given.) Might be utilized; feeds the Haggri.	NE. of Mulkalmora, Ballari.
	Haggri or Vedavatti Yerahalli	114	1198	4097	5 295	77·37	Bababudin Hills, lat. 12° 30'; feeds Ayenkaira and Maddak tank, also the Mauri Kunwal site; should be further utilized.	River Tungabaddra, 55 miles above Sunkesala.
Total of the Kistna System.	...	611	4814	6217	11 031	56·17		

[redacted]

The Pennar River System.	Feeders in Maisur.	Length in Maisur.	which the drainage is unintercepted in tanks in Maisur.	which the drainage is intercepted by tanks in Maisur.	Total area of catchment basin.	of whole area under tank system.	Rises at	Falls into
		Miles.	Sq. miles.	Sq. miles.	Sq. miles.	per cent.		
Palar ... (Falls into the Bay of Bengal in lat. 12°27'.)	Palar	47	None.	1036	1036	100·00	Chintamanipett Colar; this is entirely utilized by tanks in Maisur.	Enters Madras territory at Gumsur.
Penner ... (Falls into the Bay of Bengal in lat. 14°37'.)	Gandacholli or Jimaugal ...	60	185	452	637	77·07	Davroydrug, Tomkur; not used now, might be utilized.	
	Upper Penner ...	36	149	501	650	...	North of Nandidrug; not used, might be utilized.	
	Kushawatti ... Chittravatti ... Papakenni ...	16 23 32	None.	993	993	100·00	All rise in north of Colar Division; feed Darmavaram tank, the Kuchru tank, and Goodybunda large tank.	
Total of the Pennar system ..	...	167	334	1946	2280	85·35		
Pennar ... (Falls into the Bay of Bengal in lat. 11°25'.)	Vernshavatti ...	18	135	259	394	65·75	In Colar; not likely to be further utilized.	Joins the Penankenni.
	Penankenni ...	14	87	1060	1147	92·41	NE. of Nandidrug; feeds five large tanks, would not be further utilized.	Passes Urcottah.
Total of Pennar system ...	...	32	222	1319	1541	85·60		

ern Coast System.

THE WESTERN COAST RIVER SYSTEM.	Feeders in Maisur.	Length in Maisur.	Area over which the drainage is unintercepted in tanks in Maisur.	Area over which the drainage is intercepted by tanks in Maisur.	Total area of catchment basin.	Percentage of whole area under tank system.	Rises at	Falls into
		Miles.	Sq. miles.	Sq. miles.	Sq. miles.	per cent.		
Western Coast Rivers.	Garsappa or Sheravatti ...	49	1 101	None.	1 101	...	All rise to the west of the Ghats, are useless to Maisur, except the Sheravatti, on which are a few channels.	
	Nartravatti ...	6	780	None.	780	...		
	Puiswanni ...	12						
	Komardari ...	16						
	Other names not given	20						
Total of Western Coast System	...	103	1 881	...	1 881			
Total for the rivers of Maisur and Curg...		1606	12 777	16 287	29 064	56·16		

Brief Accounts of Indian Rivers.

The *Indus* at Attock, certain recorded velocities are as follows,

In hot seasons, opposite fort, velocity 18 miles an hour.

At tunnel site, in cold season, 5 to 7 miles an hour.

Do. in hot season, 13 to 14 miles an hour.

Surface velocity at centre, Dec. 1869, 9 miles an hour.

The rise of ordinary floods is from 5 to 7 feet in 24 hours only and is 50 feet above cold weather level. The flood of 1841 was 9 feet above cold weather level, and that of 1858, 80 feet.

Barra River, at the Lahor and Peshawar-road bridge, 7 miles w of Peshawar, the waterway allowed is 180 lineal feet. In the flood July, 1861, the flood rose 18 feet in 5 minutes, and had a surface velocity of 15½ feet per second. The soil of the bed consists, first, of 18 feet of silt and loose sand, then 8 feet of firm sand resting on clay.

Son River, Panjab, at Lahor and Peshawar Road, has a catchment area of 573 sq. miles; maximum flood depth, 15′; mean velocity, 8 to 9 feet per second; slope of bed, 14′ per mile; calculated mean velocity, 13′ per second; flood discharge, calculated from sections, 91 000 cubic feet per second = ¼″ over the catchment basin; the perennial stream is never less than 1′ deep. Bed at surface boulders; at 11′, conglomerate blocks; at 16′, a hard, dry foundation; width of river at site 1000′, but a little above only 750; clear waterway of bridge, 945 lineal feet.

The Jamna.—At the Sirsawa bridge of the Delhi Railway, 37 miles SE. of Amballa, the waterway allowed is 2376 lineal feet; at this place the Jamna is constant, for six months, from April to September, being snow-fed; it rises in March and falls in October; at the site the soil is gravel and coarse sharp sand, above the bridge site it consists of large 14 lb. boulders. Its flood velocity is 8 miles an hour, scouring the bed, carrying along the boulders and depositing them 30 feet below the ordinary bed of the river. In 1867, the river rose in flood to two feet above its banks; in 1868, 14 inches above that again.

The floods of the Jamna at Allahabad were recorded by Mr. Sibley, 1861 to 1865, observations being taken daily at 6 A.M. and he extreme variation of ordinary level within the five years' ons was 2 feet; the extreme variation of lowest level was also 2 feet. The lowest water occurred between the 19th April, when the rise from snow melting begins. The great to the periodic rains generally begins on the 19th or 20th he highest flood generally occurred between 22nd and 26th of the highest flood recorded was in 1832, a little higher than 861.

R. L. high flood 161·6, 8 days over 155 and 4 days over 160.
R. L. ... 144·5 lowest recorded flood.
R. L. ... 155·
R. L. ... 152·5

ds of 1861 were exceptionally long in duration.—The lowest flood was 30 feet above low water level, the average 40, and mum 50 feet: the maximum velocity measuring 950 feet in 81 = 12 feet per second, and for 12 days being more than 10 feet nd. At the period of greatest discharge the mean surface was 10 feet per second, and the mean sectional velocity per second; the sectional area at that level being 145 000 et, the discharge per second was 1½ million cubic feet.

iver supplies the Eastern Jamna canal with about 1065 cubic second, the Western Jamna canal with about 2500, and will ly the Agra canal with 800 cubic feet per second.

Markanda at Hassanpur, in 1859, by Mr. C. J. Campbell, C.E. ridge site, where the banks are well defined, is about three ow Hassanpor.

f channel	1 577 feet.
area	6 938 square feet.
ic slope	2·72 feet per mile.
locity	5·15 feet per second.
e	35 370 cubic feet per second.
1845	47 838 cubic feet per second.
pth	10 feet.
flood depth	6 to 9 feet.
y of bridge	1 073 lineal feet.
of roadway	24 feet above bed.
of the bed is	Sand and silt for 40 feet in depth.

k

The *Son River*, in Bengal, is 425 miles long, rising near Am Kantak in Central India, the first 325 miles of its course are rocky country; it emerges from the Kaimor hills at Rhotas, 1 miles from its confluence with the Ganges at Patna; the last 1 miles being in the plains. The river is three miles broad at Teloth and generally in the plains is two miles in breadth; for eight mont in the year the stream is a quarter of a mile broad. The extr flood discharge is said to represent 2½ inches of rainfall over the wh catchment area in 24 hours (the heavy floods never exceeding f days); in this state half the water is thrown over the country bel Massaura. The lowest d dry seasons is 4000 cubic f per second. During the referred to in the table of d charges, the rain from Jun ber inclusive was at Shahab 21·3 inches; at Bahar, at Patna, 19·6; it is genera 35 inches at each; thoug following the rainless year t fall at Patna was 50 inche.

At Dehri, a town 65 Patna, are the headworks of Son canals, and the caus Grand Trunk road. The chan of the river here varies miles in breadth, and has a of from 1·75 to 3 feet per mile, and its flood rise, or difference betwe summer and high-flood level, is from 14 to 20 feet; its discharge vari from 4000 to one million cubic feet per second. The bed is compose of shingly sand to a great depth.

It is unfortunate that the diagrams of the gaugings of this rive as well as those of the Ganges, the Kodra, the Kura, Punpun, Dur ganti, Chandarprobah, Kuramnassa, Morhar, and Sura, recorded by the engineers of the Son canals in 1872 and 1873, are not yet available.

The Ganges.—The discharges of this river given in the table obtained from Beardmore's work, were taken under the following conditions:—

1st. The quantities at Benares were taken from a section by Prinsep, on the 25th April, 1829, after a long interval without rain: the area of the section was 48 650 square feet, the width 1400, the mean depth 34·75 feet, the mean velocity 23·5 feet per minute; the maximum discharge at the same place was computed, when the river was 3000 feet wide, and had an average depth of 58 feet, and sectional area 175 000 square feet, the mean velocity being about 440 feet per minute.

2nd. The gauging at Kot, near Balliah, was taken by Lieutenant Garforth, in the first week of May, 1850, when the river was at its lowest; the sectional area was 5876 square feet, width at water-leve

25 feet, mean velocity 141 feet per minute; the maximum velocity mid-channel was 198 feet per minute, which greatly exceeded that other places where the river was deeper; the maximum depth in section was 9·42 feet in a narrow place only 120 feet wide, the mainder of the section varying from 4 to 6 feet in depth.

3rd. The gauging at Sikrigali was taken on the 9th March, 1829; this place, 30 miles above the delta, the Ganges has received the gra, the Gandak, Kusi, Son, and other rivers, whose united volume frequently more than that of the Ganges proper, Jamna, and other uents which form the river at Banaras. The data for gauging were follows: breadth about 5000 feet, depth 3 to 5 feet, sectional area 000 square feet, mean velocity about 86 feet per minute; in extreme ches the breadth is about 10 000 feet, mean depth 28 feet, sectional 280 000 square feet, the mean velocity being about 440, and the ximum 600 feet per minute.

The Ganges seems to have preserved its general course for ages wn to Suti, 34 miles below Rajmahal, where, at some period within range of tradition, some alteration in the banks caused it to be verted from its former western course, now known as the Bhagiratti far as Naddia and as the Hughli (not an indigenous name) below to its present eastern course by Rampur-Bauliah and Jellinghi ich joins that of the Brahmaputra to form the Megna estuary.

There is a lamentable want of available accurate modern information as to the physical conditions and discharges of this river.

he Damuda.—This river rises in the Sonthal Hills, its upper portion d its tributaries being comparatively unknown; it becomes a single d defined channel at about 23 miles above Raniganj, and passing ough the coalfields of that tract, enters the yellow clay of the ta near Burdwan, 52 miles below Raniganj, whence it continues to limabad. At Selimabad, 16 miles below Burdwan, is an old branch the Damuda, which flows into the Hughli above the town of that me; but the present course is by Ompta to the Hughli, opposite lta, a length of 60 miles. This river is interesting on account of floods frequently inundating the country; remedial measures, the provement of its embankments and the damming up of the old nch, were unsuccessfully attempted in 1857 by various military gineers. There is a large amount of Governmental correspondence this subject, but no valuable hydraulic data; in fact, the velocity les of the floods give as a maximum 77 feet per second, or 5 es an hour, or less than half what it must be. In 1872–73 some hydraulic observations were made by the civil engineers employ on the Orissa canals, but the records are not yet available.

The Damuda, with a catchment basin of 7000 miles, has a flo discharge representing ·125 inch per hour of rainfall.

The Mahanaddi and its Tributaries.—Reduced levels of the flo and low water sections of the Mahanaddi for the last 200 miles.

At	Miles.	Flood. Feet.	Low-wat Feet.
Sonpor	0	365·5	335·5
Barmul Pass entrance ..	60	245·5	175 5
Do exit	72	215·5	175·5
Kantilu	94	165.5	135·5
Baidessur	107	140·5	110·5
Chirchika	115	129·5	87·5
Naraj	135	92·5	65·5
Kattak	144	77·5	55·5
Mouth of Katjuri, Jaipor ..	172	37·5	15·5
Mouth of Mahanaddi ..	200	5·5	0
Mean Sea Level	...	...	0

The Tributaries of the Mahanaddi.

Torrents.	Near village of.	Distance above Kattak. Miles.	Width of Mouth. Feet.	Nature of bed.	Nature of stream.	Fall of bed per mile.
Kaligiri ...	Baidessur	37½	200	Alluvial.		
Komi	Kantilu	48¼	320	Rocky above		
Burtung...	Bentpara	64¼	300	Sandy and rocky.		6¾
Salki	Above Boad	120¼	465	Ditto.	Perennial.	
Baj	Dayah	136	700	Ditto and very rocky.	Perennial.	
Mirni	Lowpara	141	400	Sandy and rocky.		
Tel	Sonpor	143	3470	Ditto.	Perennial.	

The Mahanaddi and Katjuri have in high floods velocities of 7 fee per second. At Naraj the Mahanaddi emerges from a rocky ridg only ½ mile wide into a wide basin, 3 miles broad, and 4 miles lon reaching to Kattak. The head land of the delta at Naraj divides th Mahanaddi north of town from the Katjuri south of town. Th

Upper affluents of the Mahanaddi are in hilly country, and may be said to be unexplored.

From gaugings at Kattak it appears that the ordinary embanked channels of the delta could only carry off a flood rising to 20½ feet on the gauge, and half a flood rising to 27 feet—hence the devastation so often caused; a flood over 20½ feet may last seven days, although they remain at full height for only 12 hours. There is a sounding of 80 feet of water in the bed between Baidessur and Dewakot, being 16½ feet below mean sea-level. The Banki reservoir covers an area of 50 square miles, having a mean flood depth of 20 feet, and gives one-third of the relief from flood that is required. Total flood discharge from 27th July to 3rd August, 1855, 761 billion cubic feet; of which 645 billions can be carried off in the river channels, leaving 16 billions in 7 days = 400 000 cubic feet per second to be provided for by reservoirs, cuts, and special arrangements.

The historian of this river is Captain Harris, who laboured many years in endeavouring to mitigate the effects of its floods.

The Godavari rises at Nassik, lat. 20° 0′, long. 73° 47′, and passes south of Aurungabad, through native territory for 450 miles, until it joins the Pranhita at Sironcha. Its basin is about 60 000 square miles, or including its tributaries 120 000 square miles. Above Sironcha it is unnavigable, and had a discharge in February, 1866, of only 300 cubic feet per second. From Sironcha to Palmilla, about 38 miles, the fall of the bed is ·5 feet per mile, and this part of the river is navigable; the Pranhita having contributed a discharge of 726 cubic feet per second (Feb. 1866). From Palmilla to Enchampilli is a barrier of rock 14 miles long; known as the second barrier of the Godavari, above which the river is 1300 yards wide. From Enchampilli to Dammagudiam, 70 miles, the river has a fall of 1 foot per mile. At Dammagudiam there is a barrier of rock 8 miles long, known as the 1st barrier of the Godaveri; at this place the river is 1700 yards wide, the discharge being 1875 cubic feet per second in May, and 9375 cubic feet per second in January, having a current of 3 to 5 miles an hour. At Gollagudium, about 20 miles below this barrier, the discharge in Feb. 1866 was 2825 cubic feet per second. At Palaveram the river emerges from the hills, 80 miles below the 1st barrier, and 20 miles from the town of Rajahmandri, which is 4 miles from Dowlaishwaram, the head of the delta: for these 104 miles the fall is about ·5 feet per mile. At Palaveram the river gorge is only 200 yards wide (February, 1866), but the floods rise to 60 feet above the February level; very high freshes occur three times in the mansun and last for four or five days; the general velocity of the stream then being 6 miles an hour. The river is navigable from Sironcha downwards, excepting at the barriers, during the mansuns only, i.e. from December to May. It has three unnavigable tributaries; the Indrawatti, joining it above the 2nd barrier, which is 300 miles long, discharging 150 cubic feet per second (Feb. 1866); the Sibberi, 200 miles long, discharging 500 cubic feet per second (Feb. 1866), and joining it below the 1st barrier; and the Jal, 100 miles long.

From Sironcha to the 1st barrier the river channel has no permanence of form, it shifts [illegible] and forms large banks and shifting shoals; the banks are [illegible]e rocks that occur are sandstones and sometimes limesto[illegible] the 1st barrier to the head of the delta the channel is comp[illegible]manent, the banks are tough, the sand is large and coarse [illegible]uiring a powerful current to displace it, the rocks are m[illegible]d form natural groins, which aid in giving permanence to [illegible] From the delta head downwards the river runs in a m[illegible]ment, 6′ to 24 feet above the level of the country; its bed [illegible] per mile, the summer water surface ·7 feet per mile, and the high flood surface 1·25 to 1·50 feet per mile, down to the mouth, 40 miles below. In the delta the river, when in full flood, has a width of 2¼ miles, and a surface velocity of 4¼ miles an hour; the rise of surface varies from 20 to 50 feet; the last two feet of rise being never maintained for more than two hours. From the middle of June to the middle of September the volume is always more than 12 000 cubic feet per second; during the rest of the year 3000 cubic feet per second is considered its ordinary minimum supply. In excessively dry years the discharges have been as follows: December, 16 875 cubic feet per second; January, 8047; February, 3825; March, 2782; April, 2047; May, 1687; first half of June, 1500 cubic feet per second.

The Upper Tributaries of the Godavari, that together form the Pranhita, which is 90 miles long from Tallodhi to Sironcha, are the Warda 250 miles long, which rises in the Satpura range, and after being joined by the Wunna at the falls of Dindora, becomes navigable for the last 100 miles of its course; the Painganga, which rises in the hills south of Berar, and after an unnavigable course of 320 miles, joins the Warda above Chanda; and the Wainganga, which rises in the Satpura range north of Nagpur, takes a course of 430 miles, unnavigable, and joins the Warda at Tallodhi. The Pranhita is like the lower portion of the Warda navigable for three months in the year, from Tallodhi to Dewalmarri, where there is a barrier of rock miles long; below this to Sironcha it is navigable for four months. fall of its bed is about 1 foot per mile, so also is that of the rda in its navigable portion. Above this the Warda falls 4 feet mile, and the Wunna 2 feet per mile. The Wainganga has a fall of 6 feet in 192 miles, from Kampti to its mouth, or 2·8 feet per mile.

In 1864–67 an attempt was made by Col. Haig, aided by Captains berts and Jackson, to open a navigable communication from Din- a to the coast; it was, however, at last abandoned, on account of excessive expense.

he Kistna rises north of Sattara, Bombay presidency, in latitude , and enters the sea 35 miles SW. of Masulipatam; its catchment being 30 000 square miles. It is a perennial river 600 miles long, ering the plains at 80 miles from its mouth, and there becoming an portant river, is utilized in irrigation. In the dry weather, from vember to June, its supply is very small, being derived principally m springs in its bed; from July to October it varies much, even ling as much as 10 feet in 24 hours. In full mansun there is a stant stream 20 feet deep, the crest of its banks is from 20 to 40 t in height, and its section from 1¾ to 2¾ miles broad. At wara, the head of the delta, 60 miles from the sea, where are the t outlying spurs of the hills and the anicut or dam, the river is 00 yards wide, and has a depth in dry seasons of from 5 to 6 feet, average freshes of 31, and in highest freshes of 38 feet. In the lta it runs on an elevated ridge, having an average fall to the sea of foot per mile, varying from ·9 to 1·1 feet; the fall of the country both sides towards the sea being 1·5 feet per mile. The irrigation the delta, commenced by Captain Orr, provides for taking off 00 cubic feet per second for each side of the river, but these orks are still in an incomplete state; the irrigable area on each bank ing capable of utilizing 32 000 cubic feet per second during the son of cultivation.

The tributaries of the Kistna.

The Tungabaddra, the most important tributary of the Kistna, has a gth of about 213 miles from Gutal, where its upland tributaries, e Tunga, the Baddra, and the Choardi join the Warda, to its nction with the Kistna, at about 81 miles below Karnul. These ur upland tributaries drain an area of 3754 square miles in the ovince of Maisur, a portion of which is hilly country, having a ownpour of 135 inches, the remainder being plains with a down- ur of only 24 inches.

Of these, the Warda, draining 610 square miles, has merely a fe small anicuts on its feeders ; its ordinary mansun discharge is roughl assumed to be 5000, and its maximum flood discharge 30 000 cubi feet per second. The Haggri—joined by its affluent, the Chun Haggri, which falls into it near Mukalmuru—feeds the large Eye kaira and Maddak tanks in a comparatively rainless district, and may eventually also supply an intended large reservoir at the Mauri Kun wai pass, where its discharge has been gauged for two years, giving a an ordinary mansun discharge 4500 and as a maximum flood discharg 50 000 cubic feet per second.

The Tunga, after being join [illegible] Baddra at Kudli, is joined by the Choardi at 10 miles above [illegible], and at Harihar itself by the Sulikerri ; the maximum flo [illegible] ge of the combination of the three at the large bridge [illegible] has been determined to be 207 843 cubic feet per sec [illegible] e ordinary mansun discharg roughly calculated to be 3 [illegible]

At Wallabapur, after a [illegible] 6 miles, the Tungabaddra i joined by two tributaries, and [illegible] 0th mile by the Haggri, afte which it passes Sunkesala at its [illegible] n mile, and Karnul before joining the Kistna. At Sunkesala are the headworks of a series of canals, flowing thence to Caddapa ; and Wallavapur is the proposed site of headworks for a high-level canal, thence passing Ballari to Karnul. In order to afford further supply to these canals, it was proposed to enlarge existing reservoirs and make others on the upland tributaries of this river ; and with this view some gaugings were made on them for six months, from June to November 1865, giving the following results:—

	Sq. miles.	Million cub. ft.	Inches run off.
The Tunga, at Shemuga ...	950	229 662	108
The Baddra, at Benkipur ...	884	125 928	63
The Choardi to Maddak tank	486	54 000	50 in floods.
The Haggri, at Heriur ...	1400	1 350	
The Tungabaddra at Wallabapur		356 940	
The Tungabaddra at Sunkesala		569 700	

The proposed reservoirs on the tributaries, intended to store the above supplies, and render the present Tungabaddra canals perennial, are the Mudaba on the Tunga, the Lakkawali on the Baddra, the Masur on the Choardi, and the Mauri Kunwai on the Haggri.

Further information about the upland tributaries is given among the tabular data of the rivers of Maisur.

he Penner rises in Maisur, about 150 miles above the Madras lway-bridge, down to which point its catchment area is 4500 square es. At Perur, where its upland tributaries have joined it, the nel is larger and becomes important; from this point its course is ut 110 miles in length, without having any important tributary, to junction with the Chittravatti above Jamalmagdu, where the chment area of the latter stream is 3325 square miles: the ximum flood discharge of the Chittravatti is 23 100 cubic feet per ond, and its ordinary mansun discharge is about one-tenth of that. out 40 miles below this its tributaries the Kunder and the Papagni in it, the one having a catchment area of 3000, the other of 60 square miles: the latter has a maximum flood discharge of 14 cubic feet per second, and an ordinary mansun discharge of about -tenth of that. At 32 miles below this the Sugaler and the Cheyer it. At 18 miles below this, and at 70 miles from its debouchment o the sea, is Someshwaram, where the river leaves the Western ats, the site of the proposed headworks for a deltaic canal to irrigate Nellor side of the delta. The total length of the river from Perur the sea is about 270 miles. Its upland tributaries in Maisur are lized (see tables of the rivers of Maisur), but for the rest of its rse down to the head of the delta the river now flows on unimpeded. the Kundor, at 25 miles above its junction with the Penner, is the joli Dam and subsidiary headworks of the chain of canals from nkesala to Caddapa; the tributaries of the Kunder are also utilized the same way, affording irrigation to the large valley of the nder.

For the greater part of the year the Penner, as low even as the adras Railway bridge, is dry at the surface, though at from 1 to 4 et in the bed plenty of water can always be found. The ordinary nsun floods are 6 to 8 feet deep; the extraordinary floods, 13 feet the bridge-site the river is 1550 feet wide; the soil is clay for 5 et, gravel mixed with clay and kunkur nodules for 4 feet more, sting on a layer of sand, superimposed on hard, dark green kunkur.

The Kaveri rises in the Western Ghats, and has a catchment area, gether with its delta, of 32 000 square miles. It is fed by both nsuns, and its volume is abundant from the beginning of June to e end of December. The discharge on the 4th December, 1833, at e head of the delta, was 16 875 cubic feet per second, according to l. Cotton: but in high flood the discharge is as much as 320 625 cubic et per second. From January to May the discharge is small, much less than 16 000 cubic feet per second; though there are freshets i March and April due to local storms. Above Seringham, in Tanjor, th Kaveri divides itself into the Kaveri and the Kalerun branches, whic irrigate the delta, none of the water reaching the sea; this is due t the grand anicut of Seringham, constructed by the Telinghi rajas i remote antiquity, and restored and remodelled by Col. Cotton, betwee 1830 and 1836. The slope of the main stream above the bifucati is 3·5 feet per mile; from that to Seringham, that of the Kalerun i 2 feet per mile; from Seringham to the sea coast, its average slope i 1 foot per mile. The general [illegible] main Kaveri branch is ·4 fee per mile less than that of the [illegible] Before 1830, 12 622 cubic fee per second was utilized in ir[illegible] the Kaveri branch, and 412 cubic feet per second from [illegible]n, or 16 474 cubic feet pe second in all, out of 16 87[illegible], the works constructed by Col. Cotton, utilized 9375 c[illegible] second from the Kaveri an 7500 from the Kalerun, the [illegible]ing as much from excess a the former from deficiency. [illegible]Col. Sim made a regulatin dam across the head of the [illegible]d lowered the Kalerun da 2 feet, since when the regimen has been perfectly under control. The Kalerun is now not only a channel of irrigation, but is also the great drainage channel of the delta; the Kaveri is a channel of irrigation only, its entire volume being subdivided into small channels, and entirely utilized, although in its upper portion it is a mile in width. Information about these works is given under the head of the Kalerun deltaic canals.

The Tributaries of the Kaveri, consisting of the Upper Kaveri, the Somavatti, Hemavatti, Lachmantirth, and Lokani, join above Seringapatam. Their combined maximum flood discharge at Bannur, below that town, has been roughly determined to be 239 000 cubic feet per second; the ordinary mansun discharge, for a depth of 8 feet, is about 36 000 cubic feet per second. The other tributaries are the Kabbani, the Arkavatti, and the Shimsha; the maximum flood discharge of the Kabbani at Nanjengod is calculated to be 63 700 cubic feet per second, its ordinary mansun discharge about one-tenth of that; the maximum and ordinary mansun discharges of the Arkavatti at the Mangadi-road bridge are calculated to be 50 000 and 3500 cubic feet per second; the discharges of the Shimsha are assumed to be identical in quantity with the latter. Some further information about these tributaries is given in the data of the rivers of Maisur.

The Tambrapurni, rises in the Western Ghats, having its principal

rce in the valley of Papanassan, drains a large tract of hilly and
lland country under the influence of both mansuns, and falls into
sea south of Tuticorin. Its catchment area is 200 square miles;
course for 20 miles is in forest covered mountains, where the rain-
is from 200 to 300 inches; and for 70 miles in plains at the foot
the hills, where the rainfall is from 20 to 30 inches; for the
ainder of its course it receives a rainfall of only 18 inches. Its
at Papanassan, and that of its tributary, the Chittar, at Kurtallam,
renowned for their beauty, and are considered sacred. There are
ven native anicuts on the Tambrapurni, four on the Chittar, and two
the Mannemubuar: in addition to that now nearly constructed at
rivigantam by the English. Its floods commence in June, when
y are sometimes 10 feet deep, and frequently recur during the
t six months, or during the north-east mansun. The drainage
m the hills keeps a hot weather stream, at Strivigantam, of about
4 cubic feet per second, and never less than 198 cubic feet per
ond in March; during the six months the discharge is not less
n 600 cubic feet per second. The amount of its discharge
lized for irrigation is thus estimated in the Government re-
rds :—

	Cubic yards.
For 225 days of 1st crop at 32 cubic yds. per sec. =	237 600 000
For 45 days for 2nd crop at 15 cubic yds. per sec. =	58 320 000
For 45 days for 2nd crop at 7⅜ cubic yds. per sec. =	28 382 400

Average depth at Strivigantam 7 feet, fall 2½ to 3 feet per mile,
locity 5 to 5·6 feet per second.

The Upar.—The discharge of this stream has not been measured,
r are any observed velocities mentioned in the Madras government
cords, but its flood discharge has been thus approximated to
calculation. Its catchment area is 342 square miles, and it is
pposed that there is a maximum rainfall in 24 hours of 8 inches
er one-fourth of it, of 4 inches over another fourth, and of 2 inches
er the remainder, and that the stream carries off one-fourth of this,
ree-fourths being lost by absorption and evaporation. This gives a
od discharge of 8850 cubic feet per second.

A LIST OF THE PRINCIPAL CANALS OF INDIA.

Perennial Canals in Northern India.

FULLY DEVELOPED.

	Source.	Supply, actual or intend C. ft. per sec.
The Western Jamna Canal	The Jamna ...	2372
The Eastern Jamna Canal	The Jamna ...	1068

[illegible] LOPED.

The Ganges and Lower G [illegible] s	The Ganges ...	5100
The Bari Doab Canal	The Ravi ...	2201

UN [illegible] MODELLING.

Canals i [illegible] d in Rohilkand.

UND [illegible] RUCTION.

The Sarhind Canal	The Satlaj ...	3000
The Agra Canal	The Jamna ...	2000
The Orissa Canals	The Mahanaddi	various.
The Son Canal	The Son ...	5300
The Sakhar Canal	The Indus ...	unknown.

Inundation Canals in Northern India.

The Upper Satlaj Canals ...	aggregate length	224 miles.
The Lower Satlaj Canals ...	"	418 "
The Chenab Canals	"	222 "
The Jhelam Canals	"	unknown.
The Indus Canals in the Panjab	"	577 miles.
The Indus Canals in Sind ...	"	unknown.

Perennial Canals in Southern India.

	Supply.
The Tungabaddra Canals (not yet rendered perennial) ...	375

Inundation Canals in Southern India.

The *Deltaic* canals and anicuts of the Madras presidency.
Minor Canals in the Bombay presidency.
The anicuts and channels of Maisur.

	Capital account to 1st April, 1873.	Gross income in 1872-73.		Working expenses in 1872 73.	Percentage of profit or loss.	1872-73.		No of days the canal was open.
		Direct.	Total.			Acres irrigated.	Value of irrigated crops.	
THE PANJAB.								
Western Jamna Canal ...	£311 693	£95 362	£132 618	£40 118	31	351 820	£1 487 905	310
Bari Doab Canal	1 344 957	63 468	81 786	31 570	4	228 796	913 706	325
Upper Satlaj Canals	44 292	6 707	9 498	12 496	6	135 349	409 059	305
Lower Satlaj and Chenab ...	10 520	12 942	34 272	16 362	173	242 504	153 222	260
Indus Canals	43 736	7 866	15 960	18 046	—5	180 137	490 252	145
Shahpur Canals (Jhelam) ...	2 122	698	698	434	12	4 445	1 349	-
Total	£1 757 320	£187 043	£274 832	£119 026	8·9	1 243 051	£3 455 493	
THE NORTH-WEST PROVINCES.							Charge for interest.	
Ganges Canal	£2 605 178	£158 992	£186 660	£98 871	3·4	685 170	£116 660	244
Eastern Jamna Canal	206 177	56 253	68 561	21 918	22·7	184 154	9 310	337
Dun Canals	57 253	4 791	5 265	2 504	4·9	14 002	2 544	...
Rohilkand Canals	103 601	2 438	5 699	5 132	·7	55 650	4 121	...
Total	£2 972 209	£222 474	£266 185	£128 426	4·7	938 976	£132 635	
BENGAL.		Navigation.						
The Orissa Series	£1 221 577	£1 004	£2 295	£23 822	—1·8	4 753	£80 213	...
Midnapur and Tidal Series ..	695 812	3 544	9 805	9 621	0·	13 406	41 202	...
Total	£1 917 389	£4 548	£12 100	£33 443	—1·8	18 159	£121 415	

Financial Statistics of Canals, Punjab.

Abstract of Results on the Western Jamna Canal.

Official year.	Capital Outlay.			Working Expenses.	Direct Revenue.	Indirect Revenue.	Total Yearly Return.
	Original Works.	Establishment & other charges.	Total to end of year.				
	£	£	£	£	£	£	£
to 1821	19 443	...	19 443	332	1 136	...	1 136
1821-22	...	...	[illegible]	[illegible]	111	...	111
1822-23	...	...	[illegible]	[illegible]	−1 065	...	−1065
1823-24	...	...	[illegible]	[illegible]	— 514	...	— 514
1824-25	...	...	[illegible]	[illegible]	—2 532	12 000	9 468
1825-26	414	41	[illegible]	[illegible]	— 945	12 000	11 055
1826-27	1 814	181	[illegible]	[illegible]	—2 875	12 000	9 125
1827-28	...	...	[illegible]	[illegible]	—2 583	12 000	9 417
1828-29	...	...	[illegible]	[illegible]	— 556	12 000	11 444
1829-30	494	49	22 438	8 479	— 219	[illegible]	11 781
1830-31	...	...	...	9 337	— 733	12 000	11 266
1831-32	...	...	...	10 206	—2 118	12 000	9 882
1832-33	...	...	...	10 797	—1 186	12 000	10 814
1833-34	7 201	720	30 359	9 982	7 701	12 000	19 701
1834-35	9 028	903	40 290	10 874	3 652	12 000	15 652
1835-36	62 222	6222	108 734	4 755	9 759	12 000	21 759
1836-37	338	34	109 106	9 489	9 642	12 000	21 642
1837-38	3 174	317	112 598	10 170	19 797	12 000	31 797
1838-39	4 604	460	117 663	8 227	13 913	12 000	25 913
1839-40	626	63	118 351	9 314	16 568	12 000	28 568
1840-41	3 118	312	121 781	9 224	19 634	12 000	31 634
1841-42	1 212	121	123 114	9 520	19 937	12 000	31 937
1842-43	2 512	251	125 877	10 847	20 279	37 256	57 535
1843-44	841	84	126 802	10 314	18 785	37 256	56 041
844-45	264	26	127 092	16 927	9 104	37 256	46 361
845-46	1 718	172	128 982	14 161	15 727	37 256	52 983
1846-47	5 677	568	135 227	13 196	17 092	37 256	54 348

Financial Statistics of Canals.—Panjab.

Abstract of Results on the Western Jamna Canal.

	Capital Outlay.			Working Expenses.	Direct Revenue.	Total yearly Return.	Percentage of net Revenue on Capital.	Acreage irrigated.
	Original Works.	Establishment & other charges.	Total to end of year.					
	£	£	£	£	£	£		
48	555	55	135 838	10 539	18 529	55 786	41	
49	6 050	605	142 493	12 468	18 491	55 747	41	
50	2 087	209	144 788	14 117	17 355	54 611	38	
51	342	34	145 164	13 793	16 732	53 988	37	
52	11 248	1 125	157 537	12 648	19 855	57 111	39	
53	7 550	755	165 842	15 008	17 547	54 803	35	
54	6 871	687	173 400	12 603	21 928	59 185	38	
55	1 951	195	175 547	10 297	18 983	56 239	32	
56	984	127	176 057	12 424	21 871	59 127	34	
57	1 956	261	178 874	16 938	9 386	46 642	26	
58	491	81	179 446	10 664	12 754	50 011	28	
59	1 838	261	181 545	16 313	16 632	53 888	30	
60	2 222	330	184 096	20 317	16 816	53 573	30	
61	3 721	493	188 310	21 865	24 470	61 726	33	454 292
62	8 906	1 185	198 401	22 250	18 147	55 404	29	372 680
63	4 096	1 449	203 945	17 426	17 580	54 843	28	303 361
64	6 845	4 618	215 408	16 408	23 297	60 553	30	351 537
65	10 019	476	225 904	21 179	—5 710	31 547	15	434 964
66	903	859	227 666	20 285	28 477	65 733	20	397 963
67	446	304	228 417	23 150	34 229	71 485	31	447 171
68	1 795	364	230 577	28 711	66 313	103 569	45	331 037
69	10 716	5 696	246 989	24 102	39 574	76 830	33	486 878
70	7 939	7 955	262 884	38 979	74 405	111 611	45	496 542
71	4 816	11 474	279 173	33 878	116 884	154 140	59	462 707
72	5 780	13 084	298 036	37 645	71 651	108 907	39	444 385
73	3 454	9 895	311 693	40 118	62 182	99 438	33	351 821

Financial Statistics of Canals.—North-West Provinces.

Abstract of Results on the Eastern Jamna Canal.

Official year.	Capital Outlay.			Working Ex-penses.	Direct Revenue.	Indirect Revenue.	Total Yearly Return.	Percentage of net Revenue of Capital.	A ir
	Original Works.	Establishment & other charges.	Total to end of year.						
	£	£	■	£	£	£	£		
1823 to 1830–31	31 124	12 676	43 800						
1830–31 to 1846–47	49 074	4 907	97 761	97 522	21 454				
1847–48	1 435	149	99 360		9 503	14 965	...	...	106
1848–49	3 254	325	102 939		5 055				
1849–50	3 460	346	106 745	[illegible]	6 183				
1850–51	304	30	107 079	7 392	15 914				
1851–52	2 558	256	109 893	7 726	13 079				
1852–53	3 057	306	113 256	8 279	17 325				
1853–54	5 315	531	119 102	7 872	14 993				
1854–55	16 376	1 688	137 665	9 565	14 479				
1855–56	12 691	1 637	151 994	8 188	9 688				
1856–57	5 180	691	157 865	13 540	12 997				
1857–58	1 351	223	159 440	7 691	6 645				
1858–59	2 260	337	162 036	9 255	12 483	...	...	...	154
1859–60	393	81	162 510	10 575	20 924	...	...	...	227
1860–61	973	141	163 624	11 376	28 941	...	...	...	261
1861–62	603	3 071	167 298	11 305	22 873	...	...	...	231
1862–63	1 346	—300	168 343	8 518	25 696	3 800	29 496	13	184
1863–64	1 218	1 782	171 283	10 799	23 217	6 000	29 217	11	181
1864–65	3 366	432	174 981	12 518	36 539	6 000	42 539	18	225
1865–66	2 876	1 612	179 469	13 061	41 463	6 000	47 463	20	160
1866–67	2 844	2 269	184 582	12 247	43 131	17 769	60 900	27	239
–68	4 930	1 816	191 328	14 208	56 560	17 769	74 329	33	182
–69	4 904	1 246	197 479	15 488	50 624	17 769	68 393	28	274
9–70	2 779	282	200 539	16 508	65 728	17 769	83 497	34	251
870–71	2 324	303	203 166	18 096	60 104	17 769	77 873	30	212
1871–72	1 973	—204	204 935	19 880	61 026	17 769	68 795	24	192
1872–73	1 895	—654	206 177	21 918	56 254	12 307	68 561	23	18

[redacted]

Financial Statistics of Canals in the Panjab.

ccount of the Western Jamna Canal, to the end of 1872-73

Detail.	Previous.	In 1872-73	Total.
WORKS.	£	£	£
orary Works (to maintain pply)	...	78	78
of Land	3 316	29	3 345
nry Works.—1. Main Canal d branches	...	...	...
ams, and regulating works ...	2 487	1 017	3 504
alls and weirs	9 050	336	9 387
queducts	248	...	248
scapes	563	...	563
upply of tanks...	1 555	...	1 555
oad bridges	1 679	...	1 679
uildings	201	330	350
hwork.—1. Main Canal and anches	18 542	948	19 490
3. Drainage works	1 714	...	1 714
ellaneous.	1 312	138	1 450
l Main Canal, and branches ...	40 486	2 877	43 364
Distributing Channels.			
nry works.—*d.* Irrigation Out- s	...	576	576
enditure on general works up 1863-64	194 341	...	194 341
Total on WORKS ...	234 827	3 453	238 281
STABLISHMENT, GENERAL.			
ction	...	908	...
utive	...	4 430	...
ey	...	5 417	...
Total on ESTABLISHMENT ...	56 645	10 755	67 400
TOOLS AND PLANT.			
Total on Tools and Plant ...	1 407	19	1 426
	292 879	14 228	307 107
leduct fluctuations of suspense lance: for stock, sales, and vances ... ... Total	5 158	−572	4 586
Total CAPITAL OUTLAY. £	298 037	13 656	311 693

Financial Statistics of Canals in the Panjab.

Capital account of the Bari Doab Canal, to the end of 1872-

Detail.	Previous	In 1872-73.
WORKS.	£	£
B. *Cost of Land*...	7 333	...
C. *Masonry works.*—1. Main Canal and branches		
a. Dams and regulating works ...	75 798	155
b. Falls and weirs	137 242	6675
c. Aqueducts	17 883	...
d. Escapes	15 474	...
3. Drainage works... ...	2 478	...
5. Road bridges	103 601	94
6 Navigation works ...	18 949	...
7. Mills	1 267	...
8. Buildings	22 014	536
D. *Earthwork*		
1. Main Canal and ...	432 709	3893
3. Drainage works	7 101	...
5. Navigation Channels ...	8 193	...
E. *Miscellaneous*	65 736	46
F. *Plantations*	5 507	...
Total Main Canal and branches	921 276	11 398
Distributing Channels		
B. *Cost of Land*	3 567	...
C. *Masonry works, d,* Head sluices and regulating works	5 343	113
b. Falls and weirs	11 194	...
c. Aqueducts	14 032	...
d. Irrigation outlets	6 113	815
D. Earthwork	78 967	243
Total on WORKS ...	1 035 492	12 569
ESTABLISHMENT, GENERAL.		
Direction	...	1 761
Executive	...	11 354
Medical	...	51
Total ESTABLISHMENT ...	202 715	13 166
Tools and Plant	46 853	70
Profit and loss	4 477	...
Fluctuations of suspense-balance ..	29 592	23
Total CAPITAL OUTLAY ...	1 319 129	25 828

[redacted]

Financial Statistics of Canals in the North-West Provinces.

pital account of the Eastern Jamna Canal, to the end of 1872–73.

Detail.	Previous.	In 1872-73.	Total.
	£	£	£
WORKS.			
Main Canal.			
Masonry works. Syphon ...	...	14	14
Bridge	103	583	686
Buildings		182	182
Earthworks. Canal banks ...		49	49
Drainage works—sheds	545	44	590
Other works	158 737	...	158 737
Total Main Canal ...	159 385	872	160 257
Distributing Channels.			
The cost of these is not shown, they were made by the cultivators ...			
Masonry works		683	683
Earthwork		120	120
Escape	45	220	265
Other works	8 936	...	8 936
Total on WORKS ...	168 366	1 895	170 261
ESTABLISHMENT.			
Direction	2 328	180	2 508
Executive	26 600	250	26 850
Total on ESTABLISHMENT ...	28 928	430	29 358
Tools and Plant	621	42	663
Profit and loss	20	...	20
Fluctuations of suspense balance	7 000	—1 119	5 881
Less Receipts		—6	—6
NET OUTLAY ...	204 935	1 242	206 177
Add Simple Interest	243 272	9 310	252 582
Total CAPITAL OUTLAY ...	448 207	10 552	458 759

Financial Statistics of Canals in the North-West Provinces.

Capital account of the Ganges Canal, to the end of 1872-73.

Detail.	Previous.	In 1872-73.	Total.
WORKS.	£	£	£
Head works			
C. *Masonry works.* Weirs		2 457	2 45
Main Canals and branches ...			
B. *Cost of Land*		8	
C. *Masonry works.* Falls and		8 559	8 55
Bridges		11 894	11 6
Navigation works ...		534	53
Buildings		289	28
D. *Earthworks.* Canal em &c.		620	62
F. *Miscellaneous.* Loss on		1 557	1 55
Escapes		1 077	1 07
Drainage works ...		1 856	1 85
Other works (?) ...	1 698 817	...	1 698 81
Total Main Canal and branches ...	1 698 817	28 851	1 727 66
Distributing Channels.			
Preliminary operations		234	23
B. Cost of Land		944	94
C. Masonry works		4 570	4 570
D. Earthworks		4 155	4 15
Other works (?)	450 169	...	450 16
Total on WORKS ...	2 148 986	38 754	2 187 740
ESTABLISHMENT.			
Direction	55 081	1 615	56 69
Executive	232 302	3 866	236 16
Remodelling	16 671	...	16 671
Total on ESTABLISHMENT ...	304 054	5 481	309 535
Tools and Plant	16 725	1 473	18 19
Profit and Loss	7 101	...	7 101
Fluctuations of suspense-balance...	109 146	—17 153	91 993
Less Receipts	—9 282	— 107	—9 389
NET OUTLAY ...	2 576 730	28 448	2 605 1
Add Simple Interest	1 941 670	116 660	2 058
Total CAPITAL OUTLAY ...	4 518 400	145 108	4 663 5

[redacted]

ncial Statistics of the Deltaic Canals of Southern India.

of approximate results from remunerative works of irrigation, and channels, exclusive of tanks, in the Madras presidency.

f t.	Name of Anicut.	Up to end of 1872–73		For year 1872–73.		Percentage of net profit.
		Total Capital Outlay.	Total Gross Income.	Interest & Maintenance.	Gross Proceeds.	
		£	£	£	£	
i ...	Godavari ...	544788	3 427 377	36 023	214304	32·7
...	Kistna	358254	782 199	24 669	69303	12·5
...	Pennar ...	93395	89 142	6 200	8954	2·9
at...	Four anicuts ...	12411	32 133	743	8346	63·2
at...	Palar	21493	23 233	955	5723	
rcot	Palar	75086	34 139	3 718	2648	
Total	*Palar*	96579	57 372	4 673	8371	3·8
rcot	Poini	15420	34 987	702	641	loss
rcot	Alliabad and Cheyar	20207	24 450	1 407	2542	5·5
rcot	Vellar and nine others	52055	395 809	4 961	33321	53·8
rcot	Lower Kalerun	12974	1 106 873	2 399	41193	
......	Lower Kalerun	43974	66 118	1892	1967	
......	Upper Kalerun	24066	1 757 088	1 165	67083	
Total	*Kalerun* ...	81014	2 930 079	5 456	110243	128·3
poly	Nandiar ...	7855	9 640	406	944	6·8
tor...	Four channels...	22961	24 288	3 216	2844	loss
...	Yenamakal ...	4250	5 408	296	141	loss

–The capital outlay does not include deduction for wear and , in some instances, the cost of the distributaries. The interest cent. on the outlay up to the beginning of 1872–73.

Financial Statistics for 1864–65 of the Anicuts and Chann[illegible] of Maisur.

Division.	Rivers utilised.	Aggregate length of Channels.
		Miles.
I.—Maisur	Kaveri, Lachmantirth, Shimsha, Nugu	461
II.—Hassan	Kaveri, Himavatti, Yegachi, its branches, Shimsha	232
III.—Kaddur	Vadvutti, [illegible]risandisamudram	148
IV.—Naggar	The tribu[illegible] Tungabaddra	362
	Total ...	1203

I.—Ab[illegible] Maisur Division.

Name of Anicut.	[illegible]	Discharge.	Irrigable area at a duty of 40 acres.	Assessment due at the rate 15s. per acre.
From the Kavery.	Miles.	C. ft. p. sec.	Acres.	£
Saligram	13	40	1 600	1 200
Mirlao	40	151	6 060	4 545
Chanchamcattai ...	24	123	4 920	3 690
Tippur	22	83	3 320	2 490
Chikdeoraj	75	448	17 920	13 440
Davroi	8	73	2 920	2 190
Vijjianaddi	85	240	9 600	7 200
Bangardodi	9	90	3 600	2 700
Ramasami	31	118	4 720	3 540
Do.	30	118	4 720	3 540
Talkad	18	153	6 120	4 590
From the Lachmantirth.				
Hanagod	17	335	13 400	10 050
Kattai Malwadi ...	14	140	5 600	4 200
Harganhalli	12	150	6 000	4 500
Do.	17	224	8 960	6 720
Sagar	20	...	...	...
Cholenhalli	6	...	...	...
From the Shimsha.				
Maddur	12	56	2 240	1 680
From the Nugu.				
Lachmanpura	4	135	5 400	4 050
Total	461	2677	107 100	80 325
Averages per cubic ft. per second of discharge ..		1	40	£ 30

[redacted]

nancial Statistics for 1864–65 *of the Anicuts and Channels of Maisur—continued.*

II.—Abstract for the Hassan Division.

Name of Rivers.	Number of Anicuts.	Number of Channels.	Length of Channels.	Revenue realized in 1864–65.
			Miles.	£.
chi	...	4	15¼	472
ri	...	2	53	2010
vatti	...	8	112½	2821
ch of Yegachi ...	...	4	46	588
sha	...	1	5	19
Total ...	...	19	232	5910

I.—Abstract for the Kaddur Division, including Chikmaglur.

Names of Rivers.	Number of Anicuts.	Number of Channels.	Length of Channels.	Revenue realized in 1864-65.
			Miles.	£.
vatti	56	75	120½	3086
h	1	1	1½	23
nji	6	6	13½	340
andisamudram ...	1	...	2	7
Total ..	64	82	138	3456

—Abstract for the Naggar Division, Shemogah and Kaddur.

District.	River System.	Number of Anicuts.	Length of Channels.	Revenue realized in 1864-65.
			Miles.	£.
r ...	Sheravatti ...	46	8½	878
	Warda	22	14	
gar	Sheravatti ...	19	6½	75
idrug ...	Tunga	7	6	69
wali	Baddra	15	107¼	518
	Tunga	2	7	
b	Warda	22	17	406
arpur	Choardi	8	25½	183
	Warda	3	4	
nogah ...	Tunga	22	63	900
nahalli ...	Tungabaddra ...	3	...	22
ikerrai ...	Baddra	4	2½	5
itapur	Warda	4	5¼	135
	Choardi	4	8	
	Sheravatti ...	5	11	
tara ...	Biranji	64	77½	600
	Total ...	250	362	3791

Statistics of Irrigation from the Western Jamna Canal.

Year.	Supply admitted.	Supply utilised.	Acreage Irrigated.			Length of Distributaries.	Rainfall.
			Kharif.	Rabbi.	Total.		
	C. ft. p. sec.	C. ft. p. sec.				Miles.	Inches.
1872–73	2125	1802	202 370	149 450	351 820	The distributaries chiefly belong to the landholders.	46 to
1871–72	2147	1928	187 647	256 738	444 385		70 to
1870–71	2067	1797	218 535	244 172	462 707		48 to
1869–70	2372	...	[illegible]	62 078	496 542		31 to
1868–69	2277	...	[illegible]	86 208	486 878		31 to
1867–68	1499	...	[illegible]	44 150	831 037		31 to
1866–67	1833	...	[illegible]	86 068	447 171		68 to
1865–66	1615	...	[illegible]	01 692	397 963		37 to
1864–65	1800	...	[illegible]	37 291	434 964		
1863–64	1254	...	[illegible]	...	351 537		

The area of double crop[illegible] about 13 per cent. of the to
acreage.

Irrigating capacity varied [illegible] 30 500 acres in 1864 to 536 3
in 1871.

Mileage of canal open from 1860 to 1873—Main 102 ; branches, 31

Statistics of Irrigation from the Eastern Jamna Canal.

Year.	Supply admitted.	Supply utilised.	Acreage Irrigated.			Length of Distributaries.	Rainfall.
			Kharif.	Rabbi.	Total.		
	C. ft. p. sec.	C. ft. p. sec.				Miles	Inches.
1872–73	1050	998	79 699	104 445	184 154	625	74 to 2
1871–72	981	95 p.c.	72 404	120 345	192 749	610	114 to 2
1870–71	956	98 p.c.	98 112	114 603	212 715	608	
1869–70	...	100 p.c.	119 163	131 904	251 067	606	
1868–69	...	98 p.c.	102 141	171 960	274 101	603	
1867–68	...	94 p.c.	78 606	103 938	182 544	598	
1866–67	1068	100 p.c.	82 138	157 417	239 555	596	
1865–66	...	...	80 225	80 130	160 355	596	
1864–65	1025	...	117 770	117 770	225 266	602	
1863–64	932	...	71 129	110 202	181 331	602	
1862–63	1043	...	...	...	184 232	602	

Irrigating capacity, 1858 to 1873—250 000 acres.

Mileage of main canal, 1862 to 1873—130 miles.

[redacted]

Statistics of Irrigation from the Bari Doab Canal.

	Supply admitted.	Supply utilized.	Acreage Irrigated.			Length of Distributaries.	Rainfall.
			Kharif.	Rabbi.	Total.		
	C. ft. p. sec.	C. ft. p. sec.				Miles.	
3	1838	1208	96 718	132 078	228 796	716	
2	2073	1950	76 412	210 658	287 079	712	
1	2201	2069	88 643	190 567	279 210	710	
0	1948	1578	115 524	118 403	233 927	710	
9	1899	1649	85 519	214 315	299 834	706	
8	1532	...	106 043	156 085	262 128	696	
7	1688	...	92 699	135 753	228 452	671	
6	1431	...	91 378	84 602	175 980	623	
5	1228	...	66 370	126 313	192 683	581	
4	1340	1193	64 195	70 167	134 362	554	
3	1450	...	59 476	66 540	126 016	409	
2	1387	...	...	...	134 362		

area of double cropped land from 1870 to 1873 was 8 per
the whole acreage.

ge of canal, from 1860 to 1873. Main, 140 miles; branches,
s.

Statistics of Irrigation from the Ganges Canal.

	Supply admitted.	Supply utilized.	Acreage Irrigated.			Length of Distributaries.	Rainfall.
			Kharif.	Rabbi.	Total.		
	C. ft. p. sec.	C. ft. p. sec.				Miles.	Inches.
3	4787	4221	247 191	437 979	685 170	3228	33
2	4191	76 p.c.	232 688	373 867	606 555	3078	36
1	4300	89 p.c.	266 683	499 931	766 614	3071	38
0	5100	90 p.c.	341 846	438 560	780 406	3069	28
9	4946	94 p.c.	344 267	734 132	1 078 399	3112	16
8	3952	86 p.c.	185 137	348 319	533 456	3040	46
7	3940	89 p.c.	181 658	453 076	634 734	3039	26
6	4314	...	176 544	396 585	573 129	2777	
5	4026	...	161 835	404 682	566 517	2440	
4	4028	...	97 538	352 250	449 788	2337	
3	4850	...	90 693	114 912	205 605	2266	

ge of canal, 1862 to 1873. Main, 519 miles; branches, from
1866, 127 miles; 1867 to 1873, 135 miles.

ting capacity, 1 205 000 acres, during the above period.

Statistics of Irrigation of the Western Jumna and Bari Doab C for 1872-73.

Statement of Water utilised.

WESTERN JAMNA CANAL.				BARI DOAB CANAL.			
	Supply at head.	Discharged from escapes.	Utilised.		Supply at head.	Discharged from escapes.	
Kharif.	Cub. ft. per sec.	Cub. ft. per sec.	Cub. ft. per sec.	*Kharif.*	Cub. ft. per sec.	Cub. ft. per sec.	Cub.
1872.				1872.			
April	2359	234	2125	April ...	2198	1060	11
May ..	2523	555	1968	May ...	2208	1046	11
June ...	2446	288	2158	June ...	2146	504	16
July ...	2319	229	2090	July ...	1776	850	92
August ...	2142	562	1580	August ...	1796	768	102
September	1620	143	1477	September	1986	561	142
Average	2234	335	1899	Average	2018	798	122
Rabbi.				*Rabbi.*			
1872.				1872.			
October ...	2413	353	2060	October ...	2202	989	121
November	2540	374	2166	November	2095	915	118
December	1941	396	1545	December	1640	471	116
1873.				1873.			
January ...	1242	341	901	January ...	782	217	56
February ..	1872	249	1623	February ..	880	49	83
March ...	2084	152	1932	March ...	2342	125	221
Average	2015	311	1704	Average	1657	461	119
Average of year	2125	343	1802	Average of year	1838	629	120

[redacted]

rimate Acreage of the Irrigated Crops of the Western Jamna Canal in 1872–73.

s.	Kharif. Flow.	Kharif. Lift.	Kharif. Total.
1.			
me...	...	unknown	unknown
l ...	42 034	1 260	43 294
s 2.			
...	...	unknown	
... ...	...	...	43 143
l ...	44 281	391	44 672
s 3.			
...	90 210	5 919	96 129
... ...	...	...	305
o ...	...	...	170
nuts ...	...	...	unknown
...	...	...	15
um ...	...	...	25
... ...	...	...	114
...	6 317	172	6 489
laneous tables	...	unknown	unknown
al ...	96 646	6 261	102 907
ss 4.			
... ...	4 021	182	4 203
i ...	41	2	43
... ...	312	35	347
... ...	698	195	893
ak ...	2	...	2
wa ...	237	68	305
... ...	404	6	410
... ...	4	...	4
... ...	74	3	77
ne ...	56	85	141
... ...	28	2	30
llaneous	unknown	unknown	unknown
tal ...	6 880	4 617	11 497
ss 5.			
w ...	...	4 069	4 069
Kharif	189 840	12 530	202 370

Crops.	Rabbi. Flow.	Rabbi. Lift.	Rabbi. Total.
Class 1.			
...	...	unknown	unknown
Total ...	1 001	236	1 237
Class 2.			
...	...	unknown	unknown
Total ...	3 700	1 786	5 486
Class 3.			
Wheat ...	84 691	8 908	93 599
Barley ...	3 282	320	3 602
Oats	19	...	19
Toria	467	61	528
Tobacco ...	346	755	1 101
Poppy	5	1	6
Coriander ...	479	384	863
Methi	23	...	23
Other grains	16 848	338	17 186
Miscellaneous	...	unknown	
Total ...	115 098	10 105	125 202
Class 4.			
Masur	1 520	159	1 679
Chena	45	51	96
Charri	183	10	193
Javi	210	20	230
Lucerne ...	111	61	172
Grass	16	...	16
Miscellaneous	...	unknown	unknown
Total ...	244	17 280	17 524
Class 5.			
Gram	7 704	92	7 796
Fallow ...	...	2 231	2 231
Floodings ...	...	10 485	10 485
Total Rabbi	120 044	29 406	149 450
Kharif ...	189 840	12 530	202 370
Grand total .	309 884	41 936	351 820

.—The totals of classes are correct; the detailed acreages are evidently incorrectly classified in several instances, the crops under Classes 2 and 5 being disseminated.

Acreage of the Irrigated Crops of the Eastern Jumna Canal in 187

	Crops.	Class.	Kharif.			Rabi.	
			Flow.	Lift.	Total.	Flow.	Lift.
	Gardens	2	1 006	242	1 248		
		2	...	[illegible]	...	1 279	175
	Sugar-cane ...	1	[illegible]	[illegible]	27 426		
Cereals.	Rice ...	4	[illegible]	[illegible]	37 751		
	Maize ...	4	[illegible]	[illegible]	2 576		
	Mundwa ...	4	[illegible]	[illegible]	30		
	Sanwak ...	2	[illegible]	[illegible]	9		
	Jowar ...	2	[illegible]	[illegible]	179		
	Chena ...	4	[illegible]	[illegible]	35		
	Wheat ...	3	...	[illegible]	...	70 814	
	Oats ...	[illegible]	...	[illegible]	...	883	18 580
	Gram ...	[illegible]	...	[illegible]	...	2 379	26
	Barley ...	3	...	[illegible]	...	2 645	661
Pulses.	Urd ...	[illegible]	28	7	35		551
	Moth ...	[illegible]	3	2	5		
	Masur ...	3	...	...	...	1 274	
	Peas ...	[illegible]	...	...	...	2 146	149
	Arhar ...	3	...	...	...	2	560
Fodder.	Lucerne	2	159	24	183		...
		2	...	...	...	117	
	Dumera	2	11	...	11		27
		2	...	...	...	16	
	Charri ...	2	213	24	239		...
Fibres.	Cotton ...	3	5 936	896	6 832		
	San ...	4	3	...	3		
Drugs & Dyes.	Indigo	3	1 963	239	2 202		
		4	...	...	...	2	
	Tobacco	2	92	40	132		...
		2	...	...	...	20	
	Opium ...	2	...	...	...	2	1
	Linseed ...	3	...	...	...	51	...
	Mustard ...	[illegible]	...	...	...		1
	Waternuts ...	2	54	...	54	1 456	161
	Waste Irrigation	4	659	90	749		
		[illegible]	...	...	...	434	43
	Totals ...		72 758 83 520	6 941 20 935	79 699 104 455	83 520	20 935
	Grand Total ...		156 278	27 876	184 154		

[redacted]

reage of the Irrigated Crops of the Bari Doab Canal in 1872–73.

rs.	KHARIF.		
	Flow.	Lift.	Total.
s 1.			
cane...	8 688	470	9 158
s 2.			
s ...	225	6	231
... ...	28 829	264	29 093
'otal ...	29 054	270	29 324
s 3.			
:ds ...	780	76	856
...	24 879	2 658	27 537
... ...	169	9	178
um ...	461	25	486
laneous	1 314	457	1 771
'otal ...	27 604	3 225	30 829
ss 4.			
... ...	9 158	1 341	10 499
i ...	1 434	424	1 858
... ...	7 762	791	8 553
laneous	5 300	132	5 432
'... ...	1 049	14	1 063
'otal ...	24 705	2 702	27 407
Kharif .	90 051	6 667	96 718
Rabbi .	115 683	16 394	132 077
otal ...	205 735	23 061	228 796

CROPS.	RABBI.		
	Flow.	Lift.	Total.
Class 1.			
Sugar-cane...	11	1	12
Class 2.			
Gardens ...	117	7	124
Rice	7	...	7
Total ...	124	7	131
Class 3.			
Orchards ...	820	53	873
Wheat ...	84 866	14 751	99 417
Barley... ...	4 291	164	4 455
Linseed ...	22	3	25
Saru	731	226	957
Safflower ...	...	...	...
Poppies ...	475	58	533
Tobacco ...	56	13	69
Tukhmbalanga	...	...	...
Miscellaneous	415	262	677
Total ...	91 476	15 530	107 006
Class 4.			
Gram	7 370	78	7 448
Masur... ...	174	1	175
Sinji	14 182	686	14 868
Fallow ...	768	17	785
Miscellaneous	1 578	72	1 651
Total ...	24 072	855	24 927
Total Rabbi .	115 683	16 394	132 077

Acreage of the Irrigated Crops of the Ganges Canal in 1872-

	Crops.	Class.	Kharif Flow.	Kharif Lift.	Kharif Total.	Rabi Flow.	Rabi Lift.
	Garden produce ...	2	[illegible]	752	4 950	2 974	1 780
	Sugar-cane ...	1	[illegible]	871	68 397	19	5
Cereals.	Wheat ...	3	[illegible]	16	29	168 592	73 897
	Barley ...	3	[illegible]	26	85	77 071	75 590
	Oats ...	3	[illegible]	...	...	80	8
	Rice ...	2	[illegible]	584	26 762	124	157
	Maize ...	4	[illegible]	224	578	...	
	Jowar ...	3, 4	[illegible]	180	1 015	...	
	Chena ...	3, 4	[illegible]	222	434	40	28
	Marna ...	4	[illegible]	906	1 827	...	
	Miscellaneous	3, 4	[illegible]	997	3 150	1 701	589
Pulses.	Gram ...	3	15	3	18	13 326	3 872
	Peas ...	3	...	...	...	4 938	1 767
	Arhar ...	4	...	...	...	37	...
	Masur ...	3	...	...	...	38	15
	Miscellaneous	3, 4	11	1	18	1 728	67
Fodder.	Charri ...	4	124	2	126		
	Lucerne ...	3, 4	87	25	112	75	15
	Miscellaneous	3, 4	17	1	18	1 477	87
Fibres.	Cotton ...	3	6 722	1 239	7 961		
	San	4	199	60	259		
	Flax	3	...	...	...	114	18
	Miscellaneous	3, 4	778	11	789	146	10
Drugs and Dyes.	Indigo	3	97 267	31 513	128 780	1	...
	Miscellaneous	3, 4	586	16	602	18	3
	Opium ...	2	10	21	31	2 094	4 636
	Tobacco ...	2	76	78	154	41	77
	Miscellaneous	3, 4	...	...	...	134	162
	Oilseeds ...	3, 4	...	...	...	141	2
	Waternuts ...	2	3	...	3	1	...
	Waste irrigation ...	4	648	495	1143	144	132
	Totals ...		191 948 275 054	55 243 162 925	247 191 437 979	275 054	162 925
	Grand Total ...		467 002	218 168	685 170		

[redacted]

Brief Accounts of Indian Canals.

The Western Jamna Canal is the oldest of the perennial canals of Northern India, the most fully developed as regards its powers of irrigation, and the most remunerative. It has, however, been carried on in a most desultory manner, and even now is not complete. In 1821, the capital expended on it was £14 216, and from that time to 1833 the progress was next to nothing; in 1835, the capital account was £33 168; but in 1836, £62 225 were spent, raising it to £100 000; from that time to 1846 next to nothing was spent, the account at that date being only £119 405, according to the returns formerly given. The present capital account, given in the accompanying statistics, gives different figures, owing to an entirely new arrangement; but the same rate of carrying on the works is clearly illustrated by them. In 1853–54, this canal had arrived at a very good stage of development, after more than thirty years had been passed in spending £175 000 on works. Up to 1872–73, the capital account was £311 693, but even yet the canal has no permanent headworks, and the drainage works necessary for the healthy control of the irrigation can only be said to be commenced; and half a century has elapsed since the British first took the matter in hand.

The canal is of Musalman origin, having been projected and carried out on a small scale under the Mughal emperors. Its head is at Tajawalla, on the west bank of the Jamna, 13 miles above Dadupur; the supply being conducted from the head along an old branch of the Jamna to Bhilpur, thence by an artificial cut into the Pattrala hill torrent, and then along the latter, down to a junction with the Sombe torrent near Dadupur, where a dam and regulating head for the supply of the actual main canal are situated. After 102 miles of main canal, it divides itself at Rer, into two main branches, the Delhi branch, 75 miles long, tailing into the Jamna near Delhi, and having distributaries aggregating 100 miles in length, and the Hansi branch, which is 108 miles long to Mingnikhera, and has 141 miles of distributaries, in addition to its sub-branches. At the Joshi regulator, in the 11th mile of the Hansi branch, is the head of a sub-branch, which loses itself in the sandy desert near Rohtak after a course of 43 miles. At the 13th mile of the Hansi branch, is the head of the Butana sub-branch, 18 miles long, down to its bifurcation into two channels, one 11 the other 6 miles long.

At Minginkhera, the 108th mile of the main canal, is the head of the Bahadura sub-branch, 32 miles long, and of the Darba sub-branch, which is 18 miles long down to its bifurcation at Ramsira, whence it becomes two channels, each 10 miles long.

In addition to the various branches and distributaries, there are escape cuts from the main canal amounting to 55 miles in length, and 62 miles of escapes, cuts, and drainage lines from the Delhi branch. It is also proposed to make a new branch from the 59th mile of the main canal to Bhowani.

As regards the width of the canal, the main line varies from 360 to 120 feet, and the branches from 100 to 10; the depth is variable, the full supply depth at Dadupur being 4·3 feet, and the lowest supply about half of that,—the velocity at Tajawalla is about 17, and at Dadupur with full supply 4·14 feet per second.

The tract irrigated is 120 miles by 10.

In 1837–38, a year of famine, the acreage irrigated was 306 000, the produce saved being valued at £1 462 800; and the estimated value of the irrigated crops on 351 820 acres in 1872–73, being £2 021 811. In 1846–47, 351 501, or (360 902 ?) acres were actually watered, and the following works were completed; main canal 445 miles, excluding distributaries; bridges of various sorts, 240; main headworks, 1; stop dams, 12; aqueducts, 2; weirs and falls, 9; escapes, 4; locks, 2; irrigation outlets, 672; inlets, 36; station houses, 88; besides depôts, mills, and workshops. The gross returns in 1846 amounted to 55 per cent. on the capital. The irrigating power of water on this canal is higher than that of any canal in India, having sometimes reached nearly 300 acres per cubic foot per second of supply utilized.

While the Western Jamna canal yields the most favourable results as regards its powers of irrigation, this appears rather to be due to natural conditions than to skilful management. In 1819–20, before British reconstruction, the tract irrigated, 992 square miles, yielded £200 655 in water rate, while in 1850–51, the tract irrigated was 1015 square miles, yielding £242 177 in water rate; the increase of land revenue in each case amounting to £41 521, and the advantages due to British military management over a quarter of a century appearing very small in this particular.

The capital account of this canal was altered in the year 1863–64, by debiting it from 1820 with a share of expenses for establishment and contingencies, thus changing the sum from £190 404 to £212 989

1864 :—there is also some doubt about the establishment
ther they should be 10 or 13 per cent. on the cost of
g the whole of that period.

65 the average monthly discharge for the year was
feet per second; in the Kharif season, 1791; and in the
n, 1777 cubic feet per second.

r the value of the irrigated crops being fifty times the water
was resolved to increase the water rates, and this was
in 1867–68;—in this latter year the rainfall was excep-
urable to the cultivator, the result being that only two-
readth of wheat of the preceding year was irrigated; but
an increase of irrigation of 7436 acres of sugar-cane, the
de up.

ge of the principal irrigated crops on this canal for several
follows:—

	1860-61.	1861-62.	1862-63.	1863-64.
ne, annual ...	26 102	33 782	44 730	30 089
kharif	44 965	58 578	57 925	47 353
	43 706	33 558	25 549	45 882
abbi	181 208	148 317	111 129	145 234

	1864–65.	1865–66.	1866-67.	1867-68.
ne, annual ...	29 786	34 028	19 773	27 209
kharif	57 157	51 517	62 071	39 455
	77 738	62 684	104 796	98 800
	1 131	1 477	1 805	1 315
abbi	163 159	126 293	150 233	100 937

Col. Crofton proposed, with an estimate of £214 267, to
manent head, to complete the drainage works and the dis-
from Indri to Delhi and Jhind; it had however been dis-
1867, that the swamps near Karnal and on the Delhi and
nches were absolutely necessary; the former having existed
, in consequence of the canal from Baria to Karnal con-
cipally of natural channels.

ent state of this canal as regards works, financial condi-
rigation, is shown in the tabular statistics.

The Eastern Jamna Canal is generally very similar in cha the Western Jamna Canal;—it was constructed in about th time and the same manner, being an old, fully developed, an remunerative perennial canal: its cost was about two-thirds, average irrigated acreage about one-half of that of the latter also a restoration and enlargement of an old native work, com by the British in 1823.

The Eastern Jamna [illegible] its supply from the Jam Kharrah, and passes it [illegible] d bed of the Jamna for four to Nayashahr, where is [illegible] ing dam with 30 sluices an of the main canal. In [illegible] ten miles it crosses the m drainage at right angles [illegible] ns at each of the torrents, an continues on the high l [illegible] ountry, on the watershed b the Hindan and the J [illegible] canal is in embankment miles, its water level [illegible] 6 to 12 feet above the level country. The canal s [illegible] nsists of 130 miles of chann 625 of distributaries, w [illegible] ct 120 miles by 15.

In 1830, water was admitted through its main canal, after an e diture on works of £31 124; in 1837, the capital account had inc to £46 000, and in that year, which was one of famine, it yi £10 084 in water rate, and about the same amount in increased revenue, or in all about £20 000 or 44 per cent.; the acreage then only 96 000; the value of crops saved by irrigation was £488 eleven times the cost of the canal. In 1846–47, the capital ac was £81 460, and the acreage was 106 705, yielding £12 175 as rate, and £14 965 as increased land revenue, or as gross returns cent. on the capital. The works completed up to that time w follows:—Channels main and branch, 465 miles; irrigation o 136; dams, 11; drainage outlets, 1; aqueducts, 7; bridges, 71; and escapes, 26; falls, 14; mills, 12; workshops and station h 43.

As to the amount of irrigation effected by this canal in its stages of developement, comparatively little is known; in 1832– tract irrigated was 276 square miles, yielding £248 177 in wate and £136 742 increased land in revenue; while in 1850–51, th gated tract was 497 square miles, yielding £384 919 in water ra the same amount of increased land revenue as in 1832. A p of the canal was remodelled in 1854, and new escapes were which have since formed injurious swamps: in fact, even at p the necessary drainage works can hardly be said to have been and. From the year 1863-64 the water rates were enhanced, repairs to distributaries carried out by Government, and o maintenance; certain improvements were also effected by works. At this period, a large amount of water was usually ontract, 288 villages taking it in that manner.

reage of the principal irrigated crops grown, of which the rley, and indigo form the greatest portion of the Rabbi, or her crop, was as follows for four years: —

	1864-65.	1865-66.	1866-67.	1867-68.
r-cane, annual	28 530	29 034	20 847	26 987
... kharif	28 020	39 091	37 122	41 345
n ...	14 405	2 887	5 080	2 646
t and barley...	79 490	74 327	139 257	96 489

1-72, the gross returns amounted to nearly 30 per cent. on l. The data of the works, the finance, and the irrigation for ars will be found in the tabular statistics.

anges Canal, commenced in 1848, and opened in 1854, is the the large perennial canals of Northern India, made by the It may be considered at present to be like the Bari Doab, a loped canal, in contradistinction to the Eastern and Western anals, which have their irrigation fully developed. As it be the fate of so many Indian canals to be allowed to remain ially developed condition for a long time, their results when age are naturally interesting, although they do not admit of parison with those of completed canals.

incipal head of the Ganges canal is about 2½ miles above the own of pilgrimage, Hardwar, or Haridwar. In the first 18 its course the canal passes the Ratmu, the Ranipur and the rents, the former torrent passing through at the same level, two latter in masonry superpassages over the canal. At the , above Rurkhi, the canal crosses the Solani river in a masonry ; the embankments of approach are about 30 feet above the ad are 3 miles long; the aqueduct itself is 920 feet long, in ches of 50 feet span, and 30 feet in height. From this point the main canal nearly follows the watershed between the and the Jamna for about 181 miles to Nanun, throwing off and cuts for irrigation and navigation. From Nanun the ranch, 170 miles long, continues to Etawah, where it falls into a, and the western branch of the same length continues to

Khanpur, where it falls into the Ganges. There are also two sm
branches, 83 and 10 miles long respectively. This canal is of imm
size; it carried a supply of 5100 cubic feet per second in 1870,
utilized 90 per cent. of it; besides this it has an irrigating capacity
1 205 000 acres. As to dimensions, the first four miles from Hardw
are in natural channel, a branch of the Ganges. From Mayapur, wh
the artificial canal begins, and for a distance of 50 miles, the canal h
a constant bottom width of 140 feet, a depth of 10 feet, and a slope
bed of 1·5 feet per mile. From the 50th mile where the Fattahga
branch takes off, down to h mile, where the Bulandsha
branch takes off, the bottom is 130 feet, and the depth 9 fe
from the offtake of the Bu e branch to that of the propo
Koel branch, the bottom 110 feet, and the depth 8 fe
thence to Nanun the depth he same, but the bottom brea
varies from 96 to 80 feet attahgarh branch is at pre
83 miles long, the Buland ch 54 miles long; the Khan
and Etawah branches are in bottom width at their he
diminishing gradually to 2 eir lower extremities.

Of the details of the works as originally contemplated, there
ample given in the large work of Colonel Sir Proby Cantley, the d
signer and constructor of this canal, of whose energy, patience,
perseverance, it is impossible to speak too highly, when reflecting
the difficulties, both political as well as other, that he had to encounte

In spite, however, of the large amount of money and energy spe
upon this canal, it is a particularly unfortunate one. Its works we
once stopped for some time, owing to the caprice of a Governo
General, who wished it to be made into a purely navigation canal;
was defective in several important respects, the inclination allowed
its bed was far too high, its bed retrogressed and its falls were damage
so that it could not carry its full supply until about 1866, when a lar
additional outlay had been made. In fact, the whole of the cana
main and branches, had to be remodelled throughout; and the distri-
butaries had been so badly laid out, that hundreds of miles of the
have been abandoned at various times. The remodelling of the can
commenced in 1864, is now going on; and it is to be hoped that
will eventually carry the full supply originally intended, without in
creasing the capital account, now £2 605 178 to much beyond £3 000
While 4700 cubic feet per second is the highest amount of supply
utilized on this canal, it is probable that eventually it may rise as
as 5500, the supply for which it was originally designed and inte
being 6750 (or 7000?) cubic feet per second; should it, however,

modelling, arrive at that irrigating power, it will then have
imes the supply of the Eastern Jamna canal, at a cost of
o times as much that of the latter.
ge of the principal crops irrigated during four years was
—

	1864 65.	1865-66.	1866 67.	1867 68.
ne, annual	50 159	58 416	46 338	55 232
kharif	22 466	23 134	30 539	36 365
	42 026	10 496	19 094	5 616
... ...	35 166	47 714	70 487	75 684
nd barley	338 971	362 679	400 444	319 715

o-thirds of the irrigation effected by this canal is flush, or
the ground surface, the remainder is delivered at a low
water being raised to the surface by native mechanical
. In order to carry out the irrigation of the whole of the
ded, it is proposed to make a secondary headworks at Raj-
Ganges, and to supplement the Ganges canal by new works,
Lower Ganges canal, estimated to cost £1 825 000 in addi-
works were commenced in 1872, and £54 439 spent in
during that year.
the expenditure on works, the returns, and the irrigation
anal during late years, are given in the tabular statistics.

Doab Canal, from the Ravi in the Panjab, is the fourth of
erennial canals of Northern India.
lf-developed canal, undergoing a process of remodelling,
ery similar to the Ganges canal. It was commenced in
an original estimate of £530 000, and the greater portion
canal and works are now finished; as no account of the
rogress is forthcoming, it will be best to describe the pro-
emplated.
l is taken off from the left bank of the Ravi near Madho-
ter a length of 28 miles throws out the Kasur branch at
he 7th mile of the Kasur branch, the Subraon branch takes
wo branches will be 90 and 67 miles long respectively, the
ing into the Kasur nalla at Aljowan, the latter into the
at Subraon. The portion of the main canal from the
Kasur branch to that of the Lahor branch, which is situ-
52nd mile near Aliwal, is designated the Upper main
is 24 miles long. The remaining portion of the canal,
d of the Lahor branch to the Vahu escape, into which the

canal ta. led the Lower main branch, is 88 miles long, pass town of Amritsar, and discharges itself through the Vahn escap the Ravi. The Lahor branch from Aliwal passes Lahor, and tails the Ravi at Niazbeg, 9 miles below Lahor: its length is 59¼ mil

The section of each branch is as follows:—

	Breadth at head.		Breadth at tail.		Depths.	
	Bottom.	Mean.	Bottom.	Mean.	Highest.	Lo
Main line	112	120	112	120	4·9	2
Upper main branch ...	84	92	80	88	5·6	2
Lower main branch ...			56	63	4·6	2
Lahor branch			38	43	3·3	1
Upper Kasur branch...			60	66	4·0	3
Lower Kasur branch...			20	25	3·0	1
Subraon branch ...			20	25	3·3	1

The highest depths e with the full supply of 3 cubic feet per second, th with the lowest recorded sup of 1000: the mean widt wetted section at full supp

The mean velocity, wi ply depth of 4·9 feet, is 5·3 f per second, and that with an average depth of 4·2 feet at the canal h is 4 feet per second.

The canal is capable of irrigating 654 000 acres with full supply a duty of 218 acres per cubic feet per second.

The distributaries and escapes are as follows:—

From	Number of distributaries.	Total length.	Escapes.	Length.
		Miles.		Miles.
Main line	15	93	Malikpur ...	7
Upper main branch ...	10	75	Gulpur ...	9
Lower main branch ...	16	256	Sirkian ...	6
Lahor branch	23	291	Aliwal ...	11
Kasur branch ...	Not yet determined		Vahn ...	16
Subraon branch			Naizbeg ...	1½

In the neighbourhood of Pathankot, there are two hill torrents, the Jennah and the Chakki, which with their branches cross the line of the canal, and had to be diverted.

In 1856 it was found that the cost of the canal would not be less than £1 350 000, and work was therefore concentrated on the first 55 miles down to the Lahor branch. In 1859 water was admitted, and it was then found that, as in the case of the Ganges canal, the declivity of bed allowed was too great, the consequence being extensive channelling out in the sandy tracts and deep holes below the ls; it was also discovered that the minimum supply of the Ravi, lculated to be 2753, was actually only 1414 cubic feet per second, less than the works were designed to carry.

In 1860, a native canal, the Hasli, yielding £84 985 by direct turns, and £86 387 by enhanced land tax, was incorporated in the count of the Bari Doab canal, which then yielded nothing.

1870, or eleven years after the above-mentioned discovery, the odelling of the canal was commenced, and the Kasur and Subraon aches proceeded with, but as an additional supply from the Beas lved fresh works, the estimate of the canal and branches rose to 000 000. Progress in the remodelling was going on in 1872–73, the headworks at Madhopur were nearly completed. At present aggregate length of main canal completed is 212 out of 247 miles, d of distributaries, 692 miles. In spite therefore of everything to contrary, the irrigation from this canal in 1872 brought in a gross rn of £81 876, or a net return of £50 216, or nearly 4 per cent. on capital.

The acreage of the principal irrigated crops grown during four ars was as follows:—

		1864-65.	1865-66.	1866-67.	1867-68.
Sugar-cane, annual ...		9 878	9 181	9 156	10 600
Rice	kharif	29 212	53 564	57 615	63 691
Cotton ...	kharif	3 881	5 236	12 511	21 101
Cereals, rabbi		97 722	59 827	108 707	122 720

The estimated value of the irrigated crops grown is as follows, for everal years:—

In 1860–61, £256 024; in 1861–62, £307 238; in 1862–63, £192 668; in 1863–64, £241 969; and in 1872–73, £913,706.

Details of the works, the finance, and the irrigation from this canal re given in the tabular statistics.

The Minor Canals of the North-West Provinces.

The Dun Canals consist of five perennial canals of an aggregate length of 66 miles in the Dera Dun, a valley of the Sawalikh, or lower Himalayas, north-west of Hardwar:—they consist of—

	Miles long.	Discharge in 1872-73. C. ft. p. sec.	Supply utilized. C. ft. p. sec.	Opened in	Acreage in 1872-73. Acres.
The Bejapur ...	11	39	30	1840	5432
The Rajpur ...	12	11	9	1843	2736
The Kuttapatthar	19	33	17	1854	28
The Kattanga ...	13	25	15	1859	20
The Jakhan ...	12	15	9	1863	11

The acreage of irrigated land was not fully measured until 1867.

The distributaries have an aggregate length of 67 miles. At time the capital outlay amounted to £54 365; the direct and ind returns for that year were £3518 and £475, of which £1362 was rent, while the working expenses were £2514; in 1872–73 the expended was £57 253, the direct and indirect returns for the £4791, of which £2390 was mill rent, and £475, and the w expenses, £2504; the acreage irrigated in each of these years thus:—

	Kharif.	Rabbi.	Total.
1867–68	4334	7654	11 988
1872–73	5217	8785	14 002

The water rates were reduced in 1871, thus causing a temp loss; but in the future these canals will, after the improvem in progress are effected, yield higher returns.

The Rohilkand and Bijnaur Canals.—These consist of a numb ancient badly-designed lines, which are worked at a loss at pr though after remodelling may yield very good result:—they are

The Baigrul group	...	...	108 miles.
The Kitcha Dhora group		...	32 „
The Paha „		...	13 „
The Kailas „		...	32 „
The Nagina The Nehtor	Bijnaur		38 „

The capital outlay up to 1872–73 was £103 600; the direct, indirect, revenue and working expenses for the year £3438, £2261, and £5132 respectively; the acreage, Kharif 21 204; Rabbi 34 446; Total 55 650 acres. The length of distributaries was increased from 180 miles in 1867–68 to 294 miles in 1872–73.

The Sarhind Canal, from the Satlaj in the Panjab, is a perennial canal now under construction. It was originally projected by Sir William Baker, in 1840, the detailed project was submitted by Colonel Crofton, in 1862, and estimates for the works to the value of £2 980 427 were sanctioned early in 1872.

The headworks are at Rupar, a town at the foot of the hills. At the 38th mile (these are canal miles of 5000 feet) the main canal crosses the Grand Trunk Road, and the railway from Ludhiana to Ambala. At the 41st mile the main canal ends, and the feeder line and the combined British branches take off. The length of the combined British branches is to be 3 miles, after which they will divide

Abohar branch, 125 miles long, and the Bhatinda branch es long; the former of these will be navigable up to its 51st whence the Satlaj navigation channel will take off and after a of 45 miles tail into the Satlaj. The feeder line, which is a ation of the main line, will be divided into three sections by ads of the Kotla, Gaggar, and Choa branches of the canal, ing to native states, which take off on the right side of the line; gths of the three sections of the feeder line being 14, 16, and respectively, while that of the three branches are to be 90, 56, miles. The end of the feeder line is to be the point of on of the heads of the Choa branch and the Patiala navigation . The latter will be 6 miles long, and will tail into the Patiala near Patiala. The Choa branch will for the present tail into aggar river, although it was proposed to connect it with the rn Jamna canal by a navigation cut 55 miles long, joining it ri.

canal being partly for the benefit of native territory, one-third cost will be borne by three native states.

to the end of 1870 71, the capital account amounted to £185 667, hich half was expended in works; to the end of 1871–72, 186, of which £276 260 was on works; to the end of 1872–73, 315. of which £425 078 was expended in works, independently of lishment; of the latter sum, £240 613 was expended on about illion cubic feet of earthwork, and £107 010 on head and regu- works.

canal with its branches will be 554 miles long, and will irri- 83 000 acres in a most neglected tract of country.

Agra Canal is like the Sarhind canal, a perennial canal under uction; it will irrigate a tract on the right bank of the Jamna, en it and the Khari Naddi, from below Delhi to the Utangan below Agra.

total length of main canal is to be 140 miles, its bottom width head, 70 feet; its supply will be 1100 cubic feet per second in abbi season, and 2000 cubic feet per second in the Kharif , requiring respective depths of 7 and 10 feet. The irrigable about 1200 square miles, of which about one-tenth is uncul- e waste, and one-fifth is irrigated from wells.

supply of the Jamna at Okhla having lately been found to fall onally below 800 cubic feet per second, in May 1870 having been 72 and in January 1871 only 756 cubic feet per second, the supply of the Hind, ch is capable of giving 300 cubic feet, will also used in supplementing the canal, giving altogether 800 cubic feet as certain minimum supply, according to which the depths needful navigation are determined.

The fall of the canal from the head to the 32nd mile is ·5 feet per mile; at this point is an overfall of 5·75 feet, and beyond that to the 86th mile, the gradient is 1·0 per mile; after which it varies from ·33 to 1·00 feet per mile; below the 117th mile it becomes a simple distributary.

The discharges and velocitie are as follows:—

Mileage.		Bottom width.		Mean Velocities.	Discharges.
Head to 32	...	70'		1·82 2·36	800 min. 2000 max.
32 to 40	...	58·8		2·25 2·76	587 " 1262 "
40 to 50	...	53·4		2·29 2·88	574 " 1239 "
50 to 60	...	47·4		2·28 2·82	485 " 1044 "
60 to 70	...	41·4	4·1 6·8	2·27 2·75	429 " 910 "
70 to 80	...	30	4·2 6·8	2·26 2·69	326 " 670 "
80 to 85⅜	...	24·2	4·4 6·6	2·20 2·62	276 " 535 "
85⅜ to 95⅜	..	24·2	4·9 7·0	1·24 1·41	176 " 309 "
95⅜ to 100	...	24·2	4·8 7·0	1·22 1·41	172 " 303 "

From 100 to 117 miles the bottom widths vary from 21 to 18 feet; the depths from 3·7 to 5·2, the velocities from 1·5 to 2·3, and the discharge at the 117th mile is from 130 to 203 cubic feet per second.

The headworks at Okhla were begun at the end of 1868, and generally open in 1873; the supplementary headworks on the Hindan, below the Railway Bridge, are connected with the former by a canal having a bottom width of 24 feet, and discharging 291 cubic feet per second with a depth of 5·6 feet; it is 9 miles long, and enters the Jamna at one mile above Okhla, where there is a lock to prevent the return of flood water. The distributaries have discharges varying from 140 to 25 cubic feet per second; the principal works, bridges, escapes, and weirs are comparatively inexpensive. The total estimated cost of the Agra canal is £540 788, of which £124 200 is that of works ; the total area of irrigation is calculated at 704 000 acres, the probable net income when the irrigation is fully developed pected to be £51 375, in addition to £4000 from navigation and rent—or about 10 per cent. net.

p to the end of 1872–73, the capital account stood at £432 267, which £302 692 was incurred on account of works and plant, £73 183 on establishment, this amount having been spent in five s. Of the above outlay, £30 131 was spent on plant, £106 444 arthwork, £80 014 on falls and weirs, £37 736 on bridges, and 622 on buildings, and the remainder on miscellaneous works.

The Orissa Canals.

ies of main canals in the Orissa delta for navigation and irrigation, with headworks and distributaries.

he headworks proposed for these canals consist of three weirs ss the Mahanaddi, the Katjuri and the Beropa, 6400, 3900, and 0 feet long respectively ; the two first 12·5, and the third 9 feet h ; they are of modern design, having movable iron stanchions d shutters, that admit of being lowered to allow floods to pass over m. The canal for the irrigation of the central delta, between the hanaddi and the Katjuri, is taken off from the right flank of the hanaddi weir, and a junction canal will connect it with the Katjuri. e Taldandah canal also takes off from the right flank, and runs to ldandah, the limit to tidal navigation, and it, with its branch, the chgong canal, will eventually irrigate 155 000 acres of the central ta ; they can now irrigate 30 000, being in use for about one-third heir lengths, or 52 miles of each. Two canals are led off from the ropa weir : the one from the left bank is the high level canal, designed for navigation from Kattak to Calcutta ; of this the first 32 miles the river Brahmani are open, and the greater part of its distributaries for the irrigation of 80 000 acres are completed. the other from right flank of the Beropa weir, intended to irrigate the country ween the Mahanaddi and the Brahmani, is called the Kendrapara al ; it is 160 feet wide and 7 feet deep, and is intended to irrigate 000 acres of the northern delta, at a duty of 120 acres per cubic per second of supply ; the distributaries have an aggregate length 71 miles, and will irrigate 85 000 acres; and its Pattamandi branch ng off on the fourth mile, and running to a port on the estuary of Brahmani, will irrigate 113 000 acres.

he present estimate of the cost of these works is £2 598 200, and are intended to irrigate 1 600 000 acres.

The Midnapur canal, opened in 1871, connects Midnapur with water in the Hughli, 16 miles below Calcutta, and forms a comm cation between that river and the Kusi, Rupnarain, and Damuda. will be 52 miles long, and will effect the irrigation and drainag 200 000 acres: it is now capable of irrigating 72 000, but its distri taries and drainage channels are still incomplete. Its estimated co £931 000.

The history of the Orissa Canals is as follows:—

The preliminary designs, drawn up by Col. Sir Arthur Co in May, 1858, were estimat ost £1 300 000, and intended irrigate 2 250 000 acres. A c as granted to the E. I. Irrigati Company in June, 1861, and was raised to the amount of million as a first issue. Sr eliminary designs, and estim were drawn up afterwards ol. Rundall by May, 1863; estimate amounting to tw , and the proposed amount irrigation one and a half n , at a duty of one cubic yard hour per acre.

Certain initiatory works ated in detail, thus:—

1. Head works, comprising the Naraj weir, the Mahanaddi anicut, the Beropa anicut, and the Kattak head-works, 1500′ long × 7½′ high	£165 996
2. First Section of High-level Canal, 32 miles from the Mahanaddi to the Brahmani ...	58 449
Its distributaries, 112 miles for 87 000 acres ...	13 050
3. Kendrapara Canal, 40 miles, Kattak to False Point ...	33 537
Its distributaries, 180 miles for 270 000 acres ...	40 50
4. Midnapur Canal, 48½ miles, Midnapur to the Hughli ...	152 34
Its distributaries, 160 miles for 148 500 acres ...	22 27
5. Tidal Canal, first two reaches 27 miles from the Rupnarain	49 11
	535 26
30 per cent. for stores and management	160 58
	695 84
Surveys of general scheme, purchase of a fleet of boats, London Offices, and preliminary expenses had already cost	123 93
Interest already paid to shareholders	112 47
Total estimated cost of initiatory scheme ...	**£932 26**

stimated return. Navigation to repay establishment and manage- t, and the irrigation of 505 500 acres, at 5 Rs. per annum, to yield ross return of 36 per cent. on the £695 848, and deducting 5 per t. for repairs and maintenance, 31 per cent. net; or 21 per cent. on million of total expenditure estimated.

he works were begun in December, 1863. Irrigation was first ilable in December, 1865, was first taken up in April, 1866, and an to yield returns in October, 1866. Navigation began to yield rns in March, 1865. The Company sold the Orissa undertaking December, 1867; the works constructed and returns being as ows :—

he total amount of work done by 31st May, 1867, under the heads the preceding estimate, was—1. Headworks open, but not com- te; 2. High-level Canal, 10 miles open, 12 nearly ready, and 17 les of distributaries open; 3. Kendrapara Canal, 30 miles open, to a duced width, and 72 miles of distributaries open; 4 Midnapur nal, 28½ miles under construction, 10 nearly ready, and 46 miles of tributaries open; 5. Tidal Canal, 27 miles open without locks. ater was then available for 153 400 acres of irrigation.

Between May and December, 1867, further work was done on the ove canals, details of which are wanting, as well as 23 miles of un- pleted work on the Taldandah canal. The expenditure, up to tober, 1867, was as follows :—

Expenditure—on works up to June, 1867	...	£620 000
" ... from June to October	...	187 936
" from Oct. to Dec. 1867	...	not known.
Total expended on works in India ...	...	807 936
Total on all accounts	...	884 861
Balances	...	58 671
Receipts—not including Govt. loan of £120 000	...	£943 532

Returns from irrigation in October, 1866, and February, 1867.

At 5 Rs. ...	1067 acres and	573 acres ...	£ 821
At 3 Rs. ...	1018 acres and	2572 acres ...	1077
At 1½ and 1 Rs.	261 acres and	1183 acres ...	188
		Total, 6674 acres irrigated ...	£2086

At this time water was available for 60 000 acres.

Returns from irrigation at the end of October, 1867.

3007 acres, at 5 Rs.	... ...	£1504
1045 " " 3	... ...	313
385 " " 2	... ...	77
1799 " " 1	... ...	180
3600 " " ½	... ...	180
Total 9836 acres	... ...	£2253
Water for 13 000 acres stolen, value	... ...	£2500

At this time water was available for 153 000 acres.

Returns from navigation, beginning March, 1863.

During 1863, £376; 1864, £848; 1865, £1099; 1866, £1446; in 1867, to 21st August, £1669. · Total Navigation Returns 59

Total returns, of which £3500 was not realized ... £13 76

At the time of sale, the Company had water available for 200 00 acres, which at 5 Rs. per acre would yield £100 000, or about 10 p cent. on the total expenditure, had the cultivators taken the water; however they did not, and the Act had not then been issued (pass in February, 1870) to recover rates from land brought under wat command, it would have been unwise to extend the works, and the Company were then forced to sell up at par to the Government.

From 1867 to 1873, these works have been carried on by the Public Works Department. On the 1st April, 1873, the capital accounts stood thus:—

The Mahanaddi Project, including the Brahmani and Baitarni Series	£1 221 577
The Midnapur Project, including the Tidal Canal ...	695 812
Total	£1 917 389

The state of the works was thus in 1872-73:—

	Miles of canal completed.	Miles of distributary open per acre commanded.	Area for which water was provided.	Area for which distributaries had been constructed.	Cost per mile on original construction of canal	Cost per mile on original construction of distributaries.
			Acres.	Acres.	£	£
High-level Canal...	37	·0021	74600	42660	3018	203
Kendrapara Canal	40	·0032	313000	100000	2110	129
Taldanda Canal ...	27½	·0042	155000	15336	1398	109
Machgong Canal...	6	·0040		16829	716	95
Midnapur Canal ...	24	...	138150	69050		
Tidal Canal.........	62	...	11500	2000		

The expenditure mentioned does not include establishment nor proportionate cost of headworks.—The supply provided for the areas was at the irrigating duty of one cubic foot per second for 133 acres.

The discharge passing down the Kendrapara canal varied from 500 cubic feet per second in August, to 126 in March, and in the high-level canal from 350 in July, to 115 in March; each of the canals were closed for repair for about two months in the cold weather.

In 1869, the water rates having been lowered from 10s. to 2s. per acre, the gross revenue amounted only to £441; in 1869–70, it amounted to £5235, in 1870–71, the acreage actually irrigated was 22 128 acres; and in 1871–72 only 11 052 acres, demands for water rate being abandoned by the revenue collectors, and only £1772 being actually collected.

In the year 1872-73 the total acreage of irrigation was only 4753 acres, yielding £4263 in water rate, and the navigation returns on a tonnage of 154 422 tons amounted to £4750; the total receipts, including £1481 from various other sources, amounting to £10 293, the highest year's revenue yet obtained.

The Son Canals.—These constituting a portion of the Bahar project of Colonel Dickens, are designed to provide high-level navigation for 295 miles from Mirzapur on the Ganges through Dehri, the headworks on the Son, to Manghir on the Ganges, and to irrigate the country on both banks of the Son, between this line of navigation and the Ganges. The Western main canal, from Dehri to Mirzapur, will be 125 miles long, and will command the irrigation of an area of 2100 square miles; the Eastern main canal from Dehri to Manghir, 170 miles long, commanding 3000 square miles. The main canals are designed to carry 5300 cubic feet per second, with a depth of water of 9 feet, and a bottom width of 180 feet; in the Eastern canal the fall from the Son to the Ganges, of 123 feet, will be overcome by a series of locks. It was originally intended that these and other works should have been carried out with English capital, under the East-India Irrigation Company in 1867;—they were however commenced in 1870 by the Public Works Department, under Mr. Levinge, aided by about twenty English engineers.

The Western Main Canal was nearly completed to full dimensions for a length of 22 miles by the end of March, 1873; and its bridges and siphons were in progress. The Eastern Main Canal was then also nearly completed for eight miles. On the Arrah Canal, which is to be 70 miles long, and will irrigate 430 000 acres, ground had been broken over 60 miles; and six locks, two bridges, and seven siphons were in progress. On the Patna Canal, which will be 84 miles long, and will irrigate 390 000 acres, two-thirds of the earthwork was executed in 1872-73.

At the headworks, the masonry well blocks of the upper breast-wall of the weir were sunk right across the river in 1870-71, and in 1871-72 those of the lower breast-wall, as well as parts of the head and under sluices and head locks; the stone being brought by locomotives from quarries seven miles off.

The following is an abstract of the estimate of cost of the works:—

295 miles of high-level main canal at per mile,	£4000	£1 180 000
240 miles of main irrigation and navigable canal, at	£3000	720 000
928 miles of main irrigation distributaries ... „	£500	464 000
261 000 acres irrigated in detail ... ... „	£2	522 000
326 250 acres of minor drainage works ... „	8s.	130 500
Headworks		225 000
Workshops, shelter, &c.		43 000
		3 284 500
Superintendence at 12·5 per cent.		410 500
Tools and plant		80 000
		£3 775 000

The capital account is as follows:—

	Works and Plant.	Establishment.	Total.
Up to 1st April, 1872	£368 036	£77 456	£445 493
During 1872-73	210 951	40 635	251 587
Up to 1st April, 1873	£578 937	£118 091	£697 079

The Son weir is 2½ miles long and 8 feet high, and is especially interesting as an example of the most modern construction, exhibiting like the weirs on the Orissa canals, also designed by civil engineers, a vast improvement over everything yet done in works of this class in India. It is probable that these canals will be partly open in 1875.

The Bundelkand Canals, from the rivers Betwa and Dassan proposed by the late Captain A. H. Bagge, of the Bengal Engineers, still remain as projects under contemplation: detailed surveys were, however commenced in 1873.

THE INUNDATION CANALS OF THE PANJAB.

1. *The Lower Satlaj and Chenab Canals.*—The canals from the Lower Satlaj are 19 in number, and have an aggregate length of 418 miles; those from the Chenab are 13 in number, and have an aggregate length of 222 miles;—the whole of these, excepting 19 miles, were constructed and in working order at the time of the British annexation.

dth of these canals varies from 5 to 36 feet, and their depth of rom 3 to 11 feet; they have no distributaries, irrigation being d direct from them by means of private water-courses.

Upper Satlaj Canals are four in number:—

	Length.	Breadth.	Depth.	Distributaries.
Katora ..	66 miles	33½ feet	3·5 feet	47 miles.
Khanwah ...	81 ,,	60 ,,	6 ,,	
Upper Sohag	57 ,,	40 ,,	4 ,,	
Lower Sohag	20 ,,	20 ,,	3 ,,	

irst was constructed by the British Government, and opened in The second was constructed, for a length of 63 miles during gn of Akbar: it was reopened in 1843, and extended by the Government for 18 miles from Dewalpur southward; 25 miles ibutaries were also constructed at that time. The third was cted by the British Government, and opened in 1855; it has tributaries belonging to Government, 12 miles in aggregate and two to landholders of 16 miles, or 28 miles in all; a new completed in 1871 to serve as an alternative entrance to this or occasions when the river sets in on the old head. The was constructed by a landholder shortly after the British annex- There is also another canal, called the Nikki, about which rs are wanting.

Jhelam Canals.—There are 18 inundation canals from this the Shahpur district; they were purchased from local funds in The dimensions of two of them are as follows:—

	Length.	Mean breadth.	Average depth.
hpur Canal ...	17 miles	18 feet	6 feet.
iwal Canal ...	19 ,,	10 ,,	4·5 ,,

Indus Canals are 13 in number, and have an aggregate of 577 miles, varying from 9 to 97 miles in length; they are all from the right bank of the Indus in the Dera-Ghazi Khan at the south-western corner of the Panjab frontier: their varies from 11 to 60 feet, and their depth of water from feet; they have branches, but none of them have separate dis- channels. They were all, except one of 67 miles, the Dhundi, at the date of British annexation; but branches to the aggre- gth of 32 miles have been added since, half the expense being by the British Government, and half by the proprietors of the benefited. In addition to the above, two canals, the Fazilwah,

and the Masuwah, have been constructed and maintained by p
enterprise.

There are also some canals in the districts of Muzaffargarh, Pe
and Bannu, about which no information exists in the records.

In addition to the canals, there are a number of embank
aggregating a length of 38 miles, in the neighbourhood of Dera-
Khan, that were constructed in 1854 and 1863 for the purp
shutting out overflows in the rainy season, which used annually to
tate large tracts of country, and necessitate remissions of Gover
land-revenue.

The financial results ar ge of the Panjab Inundation C
in 1872-73, were as follo

	Capital outlay up to end of 1872-73.		1872-73.		Acreage irrigated in 187	
		D,	rect.	Working expenses	Kharif.	Rabbi.
	£		£	£		
Lower Satlaj and Chenab	10 520		830	16 362	149 143	93 361
Upper Satlaj	44 292	6	701	15 621	74 914	60 446
Indus	43 736		8 094	18 046	132 818	47 319
(average)	...	2 700	...	...	...	...
Jhelam	2 122	710	...	434	unknown.	4 445

Of the acreage irrigated by the Lower Satlaj and Chenab C
20 per cent. was lift irrigation. The mean discharge of the
Satlaj Canals was 1742, and that of the Indus Canals 4107 cu
per second in 1872. The Jhelam Canals are under the manag
of the collectors.

The Canals of the Bombay Presidency.

The Sakkar and Shahdadpur perennial canal, from the Indus i
commenced in 1861 with an estimate of £72 982, was opened in
it is 63 miles long, will irrigate 140 000 Sindian bigas of land,
expected to yield a revenue of £210 000.

The Sind Inundation Canals are of native origin, their nam
lengths are as follows:—

West of the Indus.	Head.	Length in Miles.	
The Sind ...	21 miles below Sakkar ...	66	3 br
The Ghar ...	23 miles below Sakkar ...		2 br
The Western Nara	27 miles below Sakkar ...	70	300
The Bigari ...	unknown ...	48	40 f

f the Indus.

stern Nara, Rori, improved in 1859. Acres.
itrau branch of the E. Nara, British, 190 miles, irrigates 157 000
ar branch of the E. Nara ... „ 38 000
llali ... Natural branch of Indus irrigating Haidarabad.

very doubtful whether a large proportion of these canals are
roved natural channels; there is very little information about
gation effected by them; they will probably be made eventually
e as distributaries to perennial canals, having their heads at
, at Jhirk, 250 miles below it, and at Kotri.

Jamda Canal, in Kandeish, was commenced with an estimate of
0, and was opened in 1869.

Krishna Canal has its headworks at Karwar, in Sattara, its
e was £58 133; in 1872, 32 miles of canal were finished, and
res irrigated, yielding a revenue of £955.

Ahmadnagar Canal, estimated to cost £21 941 was opened
1870.

above comprises the whole of the canals of the Bombay Presi-
Information about them is very scarce.

THE CANALS OF THE MADRAS PRESIDENCY.

Tumbaddra Canals.—The principal headworks of these canals
of a weir across the rocky bed of the Tumbaddra at Sunkesala,
et in length of clear overfall; the section varies, but is every-
8 feet broad at the top, the alternate stones of the coping being
thick, 8 feet long, and weighing each 1½ tons. The mortar used
l kankar, except for the coping which is in Portland cement.
ight varies from 6 to 26, averaging 18 feet; and the highest
d flood rose 7½ feet over the crest.

main features of the canal are as follows:—the first 75 miles
gned to carry 3000 cubic feet per second at the head, and, after
with one-fourth of this for irrigation, to convey the remainder
the Metakandal watershed cutting at its other extremity.
912·5 cubic feet per second can be discharged into the Kali,
·5 carried down the continuation of the canal. Of the 1912·5,
taken up at a fresh offtake at Jatur, and 375 at Rajoli,
750 for irrigation below Kaddapa.

minimum section of the canal in the first 75 miles has a 90-feet
breadth, with 2 to 1 side slopes. For the first 45 miles, the

fall is adapted to a maximum depth of water of 8 feet, below the 45th to one of 9 feet. The gradient of the canal is generally from ·3 to ·5 feet per mile, but in one or two deep cuttings 1·5 feet. Below the 75th mile, the natural watercourses of the Kali and the Kunda become the main channel of supply. The 1st branch channel forms the canal from the 75th to the 95th mile; it has a head sluice and lock at Lockinsula, from which it is an irrigating channel 6 feet deep for the first 6 miles, with a flow of 337·5 cubic feet per second. Below that it is a still water canal, of a minimum depth of 5 feet, and a bottom breadth of 45 feet, having a fall of 180 feet, overcome by 7 double and 5 single locks, of chambers 120 × 20; the greatest fall of a double lock being 21, and of a single one, 13 feet. The 2nd branch channel forms the canal, from the Jutur weir at the 95th mile to the 146th mile; it is adapted for a depth of 6 feet of water down to the 1st drop lock at the 118th mile. The weir is 6 feet broad at the top, on foundations of shale; it has head sluices, scouring sluices, and an entrance lock, with a water cushion below the fall. Irrigation ceases at the 130th mile. From the 118th to the 146th mile the canal consists of level reaches with 5 feet depth of water, having 17 locks to overcome a fall of 188 feet, the maximum fall in any single lock being 14 feet. The bottom breadth throughout is 50 feet. The 3rd branch channel, from the Rajoli weir at the 146¼th to the 180th mile, has also a bottom breadth of 50 feet, and with 5 feet of water will discharge 375 cubic feet per second. The Rajoli weir is made of limestone rubble, and built on rock; its top thickness is 5 feet, its front batters 1 in 2, and its lower face is vertical.

Across the Penner at Adanimayapilli are the headworks and offtake of the projected continuation of the canal to Nellor; the weir is mostly founded on wells in sand; 8 miles of this canal are open, and supply 37·5 cubic feet per second for irrigation.

The Hindri aqueduct, carrying the canal 90 feet broad, and 8 feet deep, at an elevation of 32 feet over the Hindri by fourteen 40-feet arches, is an important work. No modules are used on these canals. The ordinary hand sluices are of two sizes, one 5 feet broad, and of 3·75 feet lift, the other 1·5 feet wide, and 1 foot lift; each is worked by turning round a vertical screw that lifts a cross head, to which the cast-iron shutter hangs, each turn of the screw raising the shutter 1 inch and being easily worked in cast-iron grooves by one man against an average head of water of 6 feet.

The cost of the canal for the first 75 miles averaged £8000 a mile and for the rest of its course £2900 a mile.

This Tumbaddra project was first brought forward by Col. Haviland;
as carried out by the Madras Irrigation Company, having been
menced under the auspices of Lord Derby, and sanctioned in 1861,
estimate by Government officials amounting to one million sterling;
headworks were opened, and water admitted, in 1864: as the works
d not be completed within the estimate, a loan of £600 000 was
e to the company by the Goverment in 1866, under the condition
these works should be completed in July, 1871. They were
pleted by that date, 216 miles of canals and 377 miles of distri-
ries, commanding 91 567 acres, being opened. In 1872-73, the
age commanded was 156 570 acres, being in excess of that neces-
, when taken up, to repay the 5 per cent. interest, namely 130 000
s. The actual acreage irrigated and returns up to the present time
d thus.—

In 1870-71	1 478 acres,	yielding	£897
„ 1871-72	9 980 „	„	3541
„ 1872-73	9 505 „	„	5020
„ 1873-74	19 791 „	„	8161

The small acreage in 1870-71 was due to the damage to the canal
sed by unprecedented storms; and for which insufficient escape had
n provided. In 1871 this was repaired, and the canal improved,
d in 1872 water was again admitted throughout the whole length of
canal, to a depth of from 2 to 5 feet. In 1873-74 the canal carried
5 cubic feet per second, or 50 000 cubic yards per hour, having a
pth of 4 feet of water throughout.

The eventual irrigating power of this series of canals is assumed to
limited to 250 300 acres of rice cultivation, at a duty of 2 cubic
ds per hour per acre, in places where the waste water is lost, and
1¼ where it is again taken up by the canal; this is, however, on the
pposition that these canals remain dependent on the rainy season
pplies of the Tumbaddra; should storage reservoirs be employed,
intended, to render the canals perennial, this acreage may be
bled.

The Godaveri Deltaic Works were commenced in 1847; the head-
rks consist of a long low dam at Dauleshwaram, the head of the
lta, where the river is 6000 yards wide, from which channels are
ken off for the irrigation of the eastern, central, and western portion
the delta. The irrigable portion of the delta is 2500 square miles,
s 25 per cent. for waste land, or 1 200 000 acres. The water

available is 12 000 cubic feet per second in the flood
July, August, September, and October, and 3000 as a
during the rest of the year; the former will, at the duty of
to 1 cubic foot per second, irrigate 480 000 acres of rice, t
at the duty of 120 acres, irrigate 360 000 acres of sugar-can
two-thirds of the delta, or 840 000 acres, may be irrigated
works are completed; at present the total acreage
264 717 acres, or less than one-third.

The dam consists of several portions of masonry work r
height of 12 feet above the river bed, broken by islands
in length to 4540 feet, and connected by earthern embankme
Dauleshwaram portion is 4872 feet long, founded on wel
diameter, and sunk 6′; it is 19′ thick, consisting of a core
sand faced by a curtain wall 7′ high, and 4′ feet thick at the
a masonry counter-arched fall 28′ broad and 4′ thick; the
of cramped stone is 19′ broad and 4′ thick, the apron 80′
massive stones; on both flanks are masonry wing-walls a
ments, on the left flank a lock, head-sluices to the channel, an
sluices for silt. The Ralli portion is 2862 feet long, but ha
of rough stone. The Maddur portion is 1548 feet long;
Vegeshwaram portion 2584 feet long, having a lock and head
The earthen embankments are 7000 feet long, and the length
walls 2700 feet. The effective height of the dam may be i
by 2½ feet by means of planks held in the grooves of
standards, 6″ square and 10′ apart.

The irrigation of the eastern portion of the delta is provi
by 25 miles of main longitudinal channel, 4 miles of main tr
channel, 75 miles of main branches by Samulkotta and Corin
a series of smaller transverse channels, making on the who
intended extensions, 220 miles of main channels, from wh
village watercourses will be supplied. The supply for this
of the delta will be 4.000 cubic feet per second, or enough for
acres of rice, which is three-fourths of the culturable area.

The irrigation of the central portion of the delta is providec
the Ralli channel and its transverse lines, which amount to 9
in length, and other channels 70 miles more, in all 160 miles;
the branches of the Ralli channel crosses a minor branch
Western Godavari, in the Gannaram aqueduct, which carri
cubic feet per second, and irrigates with full supply 26 000
rice out of a culturable tract of 60 000. If this system of c
carried at full supply 4000 cubic feet per second, they would

rrigate 160 000 acres of rice and 120 000 of sugar-cane, or ll 280 000, or five-sevenths of the culturable area, 352 000

he irrigation of the western tract of the delta, is provided for a main channel breaking off into a series, having an aggregate gth of 154 miles, an extreme western channel going to the Colair with a corresponding net-work of channels will amount to 100 es, these main channels, with others of various sorts, will in all ount to 460 miles for the western tract, and will be capable of tually irrigating 280 000 acres out of a culturable tract of about 0 000.

he original estimate of Colonel Cotton for these works, in 1845, ounted to £120 000, and in 1849 this amount had been spent and original works half completed; a new estimate for £240 000 was adopted, and in 1853, £150 000 had been spent. It seems that 1849 the irrigated acreage was 127 320, yielding £41 351 gross urns, and in 1864 was 202 111, yielding £123 187 gross income, working expenses being about £26 390, and the net income 6797, or about 20 per cent. on a capital outlay up to that time, of ut £470 000.

he present financial state is shown in the tabular statement. Of progress of the works, or of the development of irrigation there is any satisfactory account forthcoming; it would appear, however, a quarter of a century has been spent in carrying out only one-rd of the intended irrigation in a district where the natives are eedingly anxious to take up water, and that the accounts are still olved in obscurity.

he Kistna Deltaic Works, designed by Captain Orr, were begun in 4. The anicut at the head of the delta at Bezwara is 3750 feet g, 305 feet broad, and has a height 21 feet above the bed of river, or 21 feet above dry season level of the water; it has er sluices on the flanks. At this point the river is 5 to 6 feet p in the dry season, and 30 to 40 feet in the mansun season; the rage flood is 24, and the highest 31 feet above ordinary low water. he dry weather, from November to June, the supply of the river o small, being principally due to springs in the bed, that the dry-on irrigation would be unimportant; in the rice season the stream ontinuous, and is 20 feet deep. The irrigable deltaic area on each k is 1 250 000 acres, requiring 31 250 cubic feet per second; each nel head however provides only 3800 cubic feet per second in the rice season, the channel having a breadth of 90 feet of water [illegible] 10 feet depth of water, and a fall of one foot per mile. The deta[illegible] of the channels are thus:—

On the right, or Gantur Bank:—	Length.	Supply.	Breadth of irrigation.	Acreage.
	Miles.	C. ft. pr. sec.	Miles.	
1st Western channel	50	1200	1½	
2nd Central channel	30	720		
3rd Eastern channel	45	1850	4½	63 200
On the left, or Masulipatam side:—				
1st channel	[illegible]	1500	3 to 4½	
2nd Drog channel ...	[illegible]	1000	2½	

It appears that there are [illegible] hole, with some others [illegible] mentioned, 290 miles of cha[illegible] the total supply during the rice season amounts to only [illegible] per second. The acreage irrigated in 1872–73 was ab[illegible]out of an acreage of 280 000 possible with the full suppl[illegible]nce assume that only three-fifths of the irrigation is u[illegible] The revenue in 1855 was £8800, and in 1863, £50 000; [illegible] financial condition is given in the tabular statistics.

All records of progress of works, and of development of irrigation on these works, are entirely wanting. At present the channels are being enlarged and widened, in order to convey enough supply for the irrigation of 430 000 acres

The Pennar Deltaic Works were commenced in 1849, and opened in 1855;—they consist of an anicut at the ferry at Nellor about 1560 feet long, and the main or Sarvaipalli channel from it, with distributaries irrigating the right tract of the delta; that on the left bank being high land is not irrigable. The supply of the Pennar being precarious, the Nellor and other tanks are utilized in keeping water in reserve and supplementing the channels. In 1857 the anicut was breached for 282 feet; and the repairs were not completed until 1861. The acreage irrigated in 1863 was 32 874; the acreage in 1872–73 is stated to be 169 073; but it is probable that this is a mistake, and includes irrigation not dependent on the anicut, more especially as the gross proceeds for the year amount only to £8954; see tabular financial results. It is now proposed to enlarge the channels, and further develop the irrigation.

The Palar Anicut and Works, in Chinglepat and North Arcot, seem to be in the same financial condition as the Pennar works; see tabular nial results. There is no official record available for ascertaining itely anything about the progress and irrigation of these works.

e *Poini, Alliabad, Cheyar, and other* anicuts in North Arcot have financial results given in the table.

e *Vellar and other anicuts* in South Arcot yield the very high net of 53 per cent. on a capital outlay of £52 055, which probably not include the whole cost of the works. There is no informa- about them available.

e *Kalerun Deltaic Works* are an improvement and enlargement of ancient native works, made under the Telingi rajahs. The grand ut of Seringham was in 1804, when Tanjor was ceded to the ish, a solid mass of rough stones, 1080 feet long, 40′ to 60′ broad, 15 to 18 feet high; this gave irrigation both along the Kalerun and Kaveri, on the former 165 000 acres, on the latter 504 900 acres, or 900 in all, which must have utilized, at the duty of 40 acres of rice ivation to 1 cubic foot per second, at least 16 747 cubic feet per ond of supply, of which 12 622 were required for the Kaveri, and 5 for the Kalerun irrigation. In point of fact, however, the total ume in December, 1833, was 16 875 cubic feet per second, of which y 9375 went along the Kaveri, and as much as 7500 along the lerun. To remedy this an anicut on the Kalerun was made be- een 1834 and 1836 by Col. Cotton; it was 2250 feet long, and thick, its height 5·3 to 7·3 feet, made of brick, capped with stone, foundations 3′ deep, built on three lines of wells 6′ deep, and 6′ xternal diameter; the apron 21′ broad, and 1′ thick, of stone in raulic cement; there were twenty-two sluices, each 2′ wide, by 3·5 h, to clear the bed of silt. In the year following its construction 240 of the dam were demolished, but were instantly repaired. In 1843 itional sluices were made, giving a total clear lineal waterway of feet, but these produced little good; and it became evident that in edying one evil, the works were causing another, the Kaveri was ly to suffer from excess of water in the same way as the Kalerun previously.

In 1845, Col. Sim made a regulating masonry dam, 1950 feet long, oss the head of the Kaveri, and lowered the Kalerun dam for a gth of 700 feet by 2 feet, this put the regimen of the Kaveri and lerun perfectly under control. The Kaveri channel is now a channel irrigation only, it is sub-divided into small channels, and its entire ume utilized; the Kalerun channel, besides giving irrigation, is the main drainage channel of the delta. The irrigation from these work is the most completely developed possible, and the returns enormous profitable; the navigation, a matter of very inferior importance i such a country, on the contrary, suffers from the dams and the si deposited above them;—in fact, a lock on the Kalerun dam had to b turned into a double sluice.

The Lower Kalerun dam was made in 1837, over the Kalerun, at 7 miles below Seringham, the head of the delta, with the followin object. At that time the Upper Kalerun dam had forced so mu water into the Kaveri, that the w [illegible] the Kalerun was much lowered, and a large amount of land [illegible] own out of water command; the object therefore was to raise [illegible] r in the Kalerun, and recove the command of it. The le [illegible] Lower Kalerun dam is 130 feet; its section consists of [illegible] wells, 6 feet deep, having sand core 3′ × 4′ in the [illegible] d over, with 4 feet of soli masonry above them for the fo [illegible] and a body wall above 7½ fee high: when the water level re [illegible] s top of the anicut, the dept of water in front is 7½ feet; [illegible] der sluices, giving 69 linea feet of waterway, and an apron in rear 24′ broad, and 3′ thick. The channel head above this dam takes off water for the irrigation of a district, eight miles below, in South Arcot: and hence, though the principal object of this lower dam was not attained by it, it has yet effected a useful purpose. In 1863 and 1864 three very serious breaches were made in this anicut, the water leaking through, and probably also, under the wells, which seem to have been carried to about half the depth necessary in such a situation, and were unprotected by any retaining wall or apron in front: it appears that in these cases the irregularity of the bed caused the current to impinge and concentrate its effects on the portions of dam that gave way.

The acreage irrigated has been materially increased, as well as saved from ruin by the former works: before 1836 it was 670 000; in 1850, 716 524; and in 1872–73, 748 673. The increase of produce effected by irrigation in these districts varies from one-fifth to one-eighth the gross produce of rice, or five to seven bushels of unhusked rice (paḍi) per acre. The Government revenue in which the water rate is merged is two-fifths the gross produce, and varies in value from nine shillings in Tanjor to twelve in Trichinopoly, and fourteen and sixpence in South Arcot, having an average over the whole of the districts of twelve shillings. The increase of annual revenue due to the works would, therefore, on 78 000 acres amount to about £47 000, while the Government returns for 1872–73 show a gross return of £110 243; see tabular statis-

It is probable, therefore, that a large portion of this latter sum is,
tly speaking, due to the works of the Telingi rajahs, constructed
re Col. Sim put the regimen of the rivers under control. If this
e case, the percentage of net profit due to the British works must
reduced from 128 to 51 per cent. on the assumed capital outlay of
014. With reference to this latter sum, it appears merely to
ude the cost of the three dams and headworks, and their recon-
ction and alterations from 1836 to 1850; if, however, we place to
capital account the cost of channels and irrigation works depend-
on these dams, which seems according to some accounts to amount
£91 874 on original works exclusive of repairs, this raises the
ital account to £172 888, and lowers the net profit to the more
sonable percentage of 24.

Apart, however, from the matter of returns, both of finance, of irri-
ion, and of works, in which it is hoped the Madras Presidency is
mencing a new era, it is an undoubted fact that the complete con-
l and utilisation of so large a river as the Kaveri, at so early a date
1846, within ten years after the original commencement of the
toration of the works, are results not known to be achieved on
y other irrigation works in the world up to the present time. They
lect lasting honour on the names of Colonels Sim and Cotton.

The Anicuts of Madura.—The Suruli, the principal tributary of
e Vaiga, joining it after a course of 36 miles from Gudalur, is
tirely utilized in the irrigation of the Kambam valley; there are
n anicuts across it, with channels and tanks; the first is situated at
lf a mile from Gudalur, whence a canal on the left bank irrigates
e lands for 5½ miles, and eventually falls into the Kambam tank:
e others irrigate a narrow strip of rice cultivation on each bank
the lower part of the Kambam valley. On the Vaiga itself are two
sonry anicuts, the Peranni and the Chitani, situated 22 and 18 miles
pectively above the city of Madura, which are said to have been
lt by two favourite dancing girls, favourites of one of the Naik
gs of Madura: the channels from them are in bad order. Below
Chitani there are no dams, the slope of the ground allowing
nnels to be taken off without the aid of anicuts. The supply of
Vaiga is so deficient in its lower parts, in the Ramnad, that any
gation from it is only on a very small scale.

The supply of the river Gundu is very small, the local rainfall
ing only 18 inches yearly; on it, east of the town of Kamudi,
miles from the sea, is an anicut large dam, made of loosely built

stone; a channel from it takes its water to the Kallavi lake. On the river Vaipar are several stone anicuts, and on its tributaries are storage tanks; the amount of irrigation effected from these two latter rivers is unknown.

The Anicuts of the Tambrapurni.—There are seven anicuts on this river. The first is the Thalay anicut, just below the falls of Papanassam, it is renewed annually with stakes and brushwood; it has two channels, one 10 miles long on the north bank, and one 6 miles long on the south, each ending in a ta cond is the Nathiani anicut, 6 miles below the former, it ncient structure, consisting of large blocks of stone pla ly across the river, and is 468 feet long; only one cha rom it, for 12 miles on the north bank, which irrigates ielding a revenue of £1297. The third is the great Ka at, built of cut stone, it is 9 feet high, and has a top w ; it has also a large rough apron varying from 35 to dth; the anicut is divided into two pieces by a rocky annel from it on the south side is 22 miles long, irrig es, and yields a revenue of £17 981; the Kannadien channel flows through the town of Serun-Mahadevi, 9 miles west of Tennevelli. The fourth is the Kodagan anicut, six miles below the last, it is 2287 feet long, of cut stone roughly put together; it has one channel from it on the north side 10 miles long, irrigating 5433 acres, and yielding £6106 of revenue. The fifth is the Palavur anicut, 2 miles east of the town of Serun-Mahadevi, it is 2532 feet long, its channel on the south side is 26 miles long, supplies 54 tanks, and terminates near Palamcotta, and irrigates 2865 acres, yielding £5468. At a mile and a half below the Palavur is the sixth or Sutamelli anicut, 2 miles east of the town of Serun-Mahadevi, divided by a rock into two portions, its channel on the north side is 14 miles long, supplying two distributaries, passing through the town of Tinnevelli, which irrigate 1806 acres, yielding £3299 of revenue.

The seventh anicut, 18 miles below the last, is the Murdur anicut, 27 miles from the sea; it is of horseshoe shape, 4028 feet long, and supplies a channel on either side; its escape weir is of beautifully cut stone work. Its channels run in and out of several large tanks, and irrigate 14 400 acres, yielding a revenue of £17 700. Below this anicut are four channels, irrigating 4280 acres, and yielding £4980 of revenue.

The total amount of irrigation effected by these native works is

578 acres, yielding £56 828; the repairs only cost 1⅓ per cent. on revenue.

The English anicut at Strivigantam, 12 miles below Murdur, will 1380′ long, 6′ high, and 7½′ broad, founded on wells; it will irrigate 000 acres on the north and 15 000 on the south bank, and supply ucorin with water; it was commenced in 1869, on an estimate of 3 160; in 1873 £76 878 had been spent on construction; it is, there-re, probably nearly completed now.

The estimated amount of water from this river that is utilized for igation is given in the brief account of the river Tambrapurni, ge [26].

THE ANICUTS AND CHANNELS OF MAISUR.

General description of Works.—The ordinary stone dam or anicut in aisur varies from 7 to 25 feet in height, it consists of a mass of dry bble, faced with large stones, placed on a rocky site; the front ing of stones 3½′ × 1⅓′ × 1′; the rear aprons of large stone blocks × 3⅓′ × 2′, each stone projecting for one-third of its length beyond t above it, or about 2⅓′; the interstices are filled with small rubble; se works are unstable and leaky, allowing all the summer discharge escape, and only supplying the channels in season of flood, when in they are easily damaged and breached; the dams are curved and int up stream, having a length about double the width of the river, crown is lower near the head-sluices to relieve the pressure against m in flood. The head sluices consist of rough stone uprights, 4 or feet apart with stone caps over them; the openings being stopped ith brushwood or earth filling; they are very inefficient during floods, hich frequently enter uncontrolled and make breaches.

The channels are rough trenches generally following the undulation the country, and very badly levelled and set out; the irrigation ater is taken direct from them through cuts made in their banks, the capes for surplus water are made in the same way; the channels ffer much from silt brought down by cross drainage, also from breach-g by the same cause; although there are rough stone silt dams as ell as solidly constructed outlets at low levels for holding up and ouring out the silt from the channels.

Results.—The financial results, as shown in the tabular statistics, ppear meagre in the extreme; the causes being that not half the rigated land is assessed, and that the irrigation water is surrepti-iously taken. It appears that if all the irrigation were paid for, the nks *of the Maisur division* would yield £56 900, and those of the

[redacted]

Large Districts.	Description.	Height above sea.	Catchment area.	Mean Discharge of year.	Mean. Discharge per sq. mile	Representing rainfall run off.	Registered rainfall per annum.	Maximum discharge per square mile.
		Feet.	Sq. miles.	c. f. p. s.	c. f. p. s.	Inches.	Inches.	c. f. p. s.
Bann and Lough Neagh, at Weir..	Hilly	46 to 1765	22 205	8319	1·58	21·44	27·44	5·00
Brosna, at Ferbane Bridge.........	Hilly	152 to 1054	446	736	1·65	22·38	36·70	8·91
Robe, Mayo, at Ballinrobe	Flat	100 to 370	109	235	2·16	29·14	49·25	17·62
Loch Lubnaig	Precipitous ..	400 to 2500	70	417	5·97	81·33	70·6	78·63
Loch Katrine	Precipitous ..	400 to 2500	72	436	6·09	81·70	103·3	34·92
Small Hill Districts.								Storage. Millions of cubic feet.
Bann Reservoirs .... 1838 ...	Hilly	400 to 2800	5·15	18·21	3·50	48·	72·	56·0
Greenock .... 1828 ...	Flat Moor ...	512 to 1000	7·88	23·61	3·29	41·	60·	38·
Glencorse Pentland	Precipitous ..	734 to 1600	6·00	10·	1·66	22·3	37·	7·66
Belmont .... 1843 ...	Moorland ...	850 to 1600	2·81	10·5	3·74	50·7	63·4	26·8
Rivington Pike .... 1847 ...	Various	800 to 1545	16·25	48·0	2·94	40·	55·5	29·6
Turton and Entwistle ... 1836 ...	...	500 to 1300	3·18	9·61	3·02	41·	46·2	31·43
Ashton .... 1844 ...	...	800	0·59	60·78	1·09	15·5	40·0	21·0
Bute .... 1826 ...	...	200 to 350	7·80	13 65	1·75	23·9	45·4	...
Bolton	...	800 to 1600	0·80	1·67	2·09	32·7	46·0	25·6
Brockburn, Glasgow ... 1852 ...	...	400 to 800	4·30	3·49	0·79	47·4	60·4	...

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* The storage effected by these reservoirs is in millions of cubic feet per square mile of catchment.

ata of Reservoirs constructed in various Countries. (Heywood.)

	Name of Reservoir.	Description of dam.	Surface.	Contents.	Maximum depth of water.	Maximum height of dam.	Cost.	
			Square miles.	Millions of cubic feet.	Feet.	Feet.	£	
France	Grosbois	Masonry.	...	300	...	...	144 000	Dupuit.
	Canal de Bourgogne	„	...	706	...	...	480 000	„
	Furens	„	...	56·5	164	...	64 000	Graeff.
England	Woodhead	Earth.	0·21	198	[illegible]	[illegible]	...	Bateman.
	Torside	„	0·25	236	[illegible]	[illegible]	...	„
	Rhodeswood	„	0·08	80	[illegible]	[illegible]	...	„
	Arnfield	„	0·06	33·6	52	67	...	„
	Hollingworth	„	0·02	11·7	52	70	...	„
India ...	Muta	Masonry.	5·25	715·	83	...	95 000	Col. Fife.
	Sholapur	„	6·33	2222	60	...	54 300	Campbell.
	Cholavaram	Earth.	...	3602	...	...	64 000	Fraser.
	Vahar	„	2·17	1732	80	84	...	Conybeare.
	Eight tanks in Rajputana	Earth and masonry.	...	624·3	15 to 27	...	8 114	Col. Dixon.
	Nagpur	Earth.	0·58	257·5	45	81	40 000	Binnie.

[redacted]

			pressure (P).	tion* for P=6.			
	Metres.	S. metres.	Kilogrammes per s. centimetre.	S. metres.		Inner.	Outer.
Puentès	50	1519	7·9	1029	Rubble, faced with ashlar	Perpendicular	Stepped.
Alicante	41	1100	11·8	566		Sloped ...	Stepped.
Val de Infierno ...	35·7	1084	6·5	391		Perpendicular	Stepped.
Nijar	27·5	499	7·5	308		Perpendicular	Stepped.
Elche	23·2	213	12·7	187		Sloped ...	Sloped.
Almansa	20·7	139	14·0	141		Perpendicular	Stepped.
Furens	50	1029	6·0	1029	Rubble ...	Curved ...	Curved.
St. Chamond ...	42	...	7·9	...	Rubble ...	Curved ...	Curved.
Bosmeleac	16·8	...	8·4	...	Rubble, faced with ashlar	Perpendicular	Sloped.
Glomel	13·9	...	9·2	...		...	...
Grosbois	27·1	...	14·3	...		Stepped ...	Sloped.

* *N. B.*—The theoretical section is that of the Delocre type.

Spanish Dams: Details of Scouring Galleries and Inlet Wells. (Heywood.)

Name of Dam.	Size of scouring gallery.						Method of closing the entrance.	Dimensions and shape of inlet well.		Inlet openings.				Distance from edge of dam.
	Up stream side.		Middle.		Down stream side.			Up stream side.	Down stream side.	Vertical distance apart.	Horizontal distance apart.	Size.	No in a tier.	
	Height	Breadth	Height	Breadth	Height	Breadth								
Alicante ...	8·56	5·9	9·84	10·82	19·18	13·12	Wooden door	Curved			0·98	·86 × ·72	2	1·968
Elche	6·56	5·9	...	...	10·83	7·58	Do.	Do. 3·0	8′ Do.	...	...	·54 × ·75	1	1·5
Almansa ...	4·92	4·26	..	...	4·92	4·26	Do.	...	...	...	...	...	...	...
Puentès ...	...	...	...	...	24·70	21·97	Do.	Curved 13·77	8 2 Rectangular	2 72	...	·92 × 1·8	3	...
Val de Infierno ...	14·70	9·62	...	...	14 70	12 30	Two wooden doors.	Curved chord of 14′ Rectangular 8 75	Rectangular 14′ × 5 3 8·20 Rectangular	0·84	...	...	2	11 8·03
	[illegible]	[illegible]			7·18	3 28	Sluice gate	Circular 8·	·92′ Circular	1·47	...	...	1	11·77

Year.	Capital outlay.	Direct income.	Increased land revenue.	Gross returns.	Working expenses.	Net returns.	Interest charges.	Acreage irrigated.			Rainfall.
	Total results up to the end of each year.							During the year.			
								Kharif.	Rabbi.	Total.	Inches.
1872-73	£18 338	£4684	£50 079	£54 763	£19 557	£35 206	£20 182	2108	8979	11 087	20 to 30
1871-72	18 338	4246	47 626	51 872	18 460	33 412	19 357	1857	5937	7 794	8 to 28
1870-71	18 338	3712	45 173	48 885	17 308	31 577	18 464	1306	7085	8 391	13 to 19
1869-70	18 338	3238	42 720	45 958	15 529	30 429	17 547	1580	8166	9 746	13 to 23
1868-69	18 255	3048	40 267	43 315	14 299	29 017	16 634	1021	2042	3 063	11 to 20
1867-68	17 530	2419	37 815	40 234	13 349	26 884	15 757	5347	8253	13 600	20 to 37
1866-67	17 010	2269	35 362	37 631	12 823	24 808	14 907	1620	7552	9 172	17 to 41
1865-66	17 010	2122	32 909	35 031	12 069	22 963	14 056	1943	5504	7 447	
1864-65	16 914	1776	31 342	33 118	10 337	22 781	13 211	738	2057	2 795	
1863-64	16 791	1453	29 775	31 227	8 965	22 262	12 371				
1862-63	15 251	1102	28 208	29 310	7 905	21 405	11 609				
1861-62	15 251	1006	26 641	27 646	7 348	20 298	10 846				
1860-61	15 251	721	25 074	25 795	7 137	18 657	10 083				
1859-60	14 843	694	23 507	24 201	6 916	17 285	9 341				

Abstract of Financial Results of the Bundelkand Irrigation Reservoirs.

Year.	Total results up to the end of each year							During the year.			
	Capital outlay	Direct income	Increased land revenue	Gross returns	Working expenses	Net returns.	Interest charge.	Acreage irrigated.		Free irrigation	Exclusive of free irrigation
								Kharif.	Rabbi.		Total.
1872-73	£7203	£1318	£3214	£4532	£4278	£254	£1532	810	1300	Amount unknown.	1010
1871-72	7105	1099	2959	4058	3732	326	1019	806	877		1183
1870-71	7105	934	2704	3638	3281	[illegible]	[illegible]	[illegible]	829		946
1869-70	6550	746	2449	3196	2685	[illegible]	[illegible]	[illegible]	781		962
1868-69	5893	540	2199	2748	1756	[illegible]	[illegible]	[illegible]	998		1340
1867-68	3033	401	1933	2334	1400	[illegible]	[illegible]	[illegible]	598		793
1866-67	2291	288	1600	1889	986	903	13	178	715		803
1865-66	290	186	1297	1484	573	910	...	65	564		629
1864-65	...	...	1002	1002	136	866					
To end of 1864	...	...	...	...	15						

N.B.—These works having been under the charge of the collectors until lately, the correct financial condition—not even the extent of irrigation—cannot be arrived at. The above may be considered as a very rough approximation to, or indication of, real state of the works.

NAME OF TANK.	Surface of tank when full.	Mean depth of whole tank.	Contents of tank when full.	Area irrigated from tank.	Amount of storage irrigated per acre.	Gross revenue due to tank.	Gross revenue per acre irrigated.	Gross value of one million cubic feet of water in tank.	Capital expended.	1866–67. Balance of		1866–67. Net percentage on capital.	
										Income.	Charge.	Gain.	Loss.
	Sq. ft.	Feet.	Cub. ft.	Acres.	Cubic feet.	£	£	£	£	£	£	£	£
1. Lusani ...	9 525 600	8·0818	76 984 185	192·09	400 771*	141	·734*	1·831	1118	762	...	6·85	...
2. Dewatan ...	6 350 400	6·8045	40 036 224	303·00	132 133	158	·521	3·950	1330	853	...	6·41	...
3. Kabra ...	7 938 000	...	57 693 384	57·96	995 400*	112	1·933*	1·943	767	660	...	8·60	...
4. KaliKankar	7 938 000	7·268	55 932 220	339·32	175 409	144	1·424	2·572	2479	124	...	0·50	...
5. Darathu ...	17 424 000	7·3308	127 836 403	448·14	285 260	147	·327	1·376	3171	...	291	...	0·92
6. Niran ...	13 939 200	7·6216	106 239 286	913·94	116 243	560	·612	5·271	3208	3865	...	12·05	...
Total...	...	...	..	...	709 945	...	1·884	16·943	12068	[illegible]	291	...	...
	...	...	...	...	...	...	..	...	...	5973	...	...	...
Averages...	...	...	...	...	177 261	...	·471	2·824	...	...	...	4·96	...

N. B.—Figures marked thus (*) in columns 5 and 7 to be omitted in striking averages; the area irrigable from the Lusani and Kabra tanks being very small in comparison with their cubic contents.

Data of the Maisur Tank System.

MAISUR RIVER SYSTEM.	Total length of main rivers with their affluents.	Amount of drainage area unintercepted by tanks.	Amount of drainage area intercepted by tanks.	Total area of each catchment basin.	Proportion of whole area under the tank system
	Miles.	Sq. miles	Sq. miles.	Sq. miles.	Percentage
I. Kistna River ..	611	4 814	6 217	11 031	56
II. Palar ...	47	[illegible]	1 036	1 036	100
III. Pennet ...	167	[illegible]	1 946	2 280	85
IV. Pennar ...	3	[illegible]	1 319	1 541	85
V. Kaveri... ...	64	[illegible]	5 769	11 295	51
VI. Western Coast rivers ...	[illegible]	[illegible]	...	1 881	
Totals for Maisur and Curg . .	1606		16 287	29 064	56
Deduct for Curg .	...	1 795	...	1 795	
Total for Maisur only .	1516	10 982	16 287	27 269	60

Data of the Maisur Tank System—continued.

MAISUR TANK SYSTEM.	Under wet and garden cultivation.	Expenditure on repairs other than the Astagram channels.	Average yearly outlay.
	Acres.	£	£
From 1837–38 to 1841–42 ...	1 705 150	47 018	9 404
From 1842–43 to 1846–47 ...	1 849 759	43 225	8 645
From 1847–48 to 1851–52 ...	2 087 929	58 644	11 729
From 1852–53 to 1856–57 ...	2 160 309	70 021	14 004
From 1857–58 to 1861–62 ...	2 169 040	80 762	16 152
25 years' total outlay	...	299 670	11 987
25 years on channel repairs ...	...	57 537	2 301
25 years on tanks only ...	...	242 133	9 685

BRIEF ACCOUNTS OF INDIAN RESERVOIRS.

he *Delhi and Gurgaon Irrigation Works.*—These works, consisting kes and reservoirs, have for their object the irrigation of the try south of Delhi, and in the Gurgaon and Rohtuk districts, a t deal of which is broken by small ranges of low hills. Attention directed to these districts by the fearful famine of 1860, and the ernment of the Panjab then ordered that works should be comced to relieve the fearful destitution and starvation then existing; country was therefore examined, and surveys and designs made by L. D'A. Jackson, then assistant engineer in sole charge, for the struction of storage reservoirs in the Gurgaon and neighbouring ricts. The larger reservoirs and artificial lakes in the Delhi dists, originally constructed by the Mughal emperors, Akbar, Firoz h, Aurang Shah, and Firoz Toghlak, have been reconstructed and ewed since British occupation.

he natural basins are:—

. The Najafgarh Jhil, filled by the Sahiba and its affluents.

. The combined Kotila, Chandni, Malab, and Rajira Jhils.

hese collect the drainage of the surrounding country, and saturate land submerged; the water is then drawn off by escape channels, d the beds of the jhils are cultivated. The superintendence of se works was originally under Mr. Batty.

The artificial reservoirs, twenty-four in number, are formed by banking natural ravines, or outfalls of natural lines of drainage; y have weirs and escape channels; irrigation is thus given to the ds above the embankment, which are cultivated after submersion, d to lands below by means of the supply given through the annels. The names of these reservoirs are:—

In the Delhi District.

1. Tilpat.
2. Palam.
3. Yahia Nagar.
4. Chattarpur.
5. Khirki.
6. Naryanah.
7. Toghlakabad, No. 1.
8. Toghlakabad, No. 2.
9. Bijwasan.
0. Aurangpur.
1. Ambarheri.
2. Badli.

In the Gurgaon District.

1. Tharsa.
2. Gwalpahari.
3. Ghatta.
4. Pattri Katal.
5. Kala.
6. Raisinah.
7. Bar Gujar.
8. Dahina.
9. Nand Rampur Bas.
10. Bahari.
11. Jhand Sarai.
12 Garhi Harsaru.
13. Banarsi.

Both the jhils and the storage reservoirs are entirely depende their supply on the annual rainfall, and many of them being the loss from evaporation is very great: unfortunately also, seve the reservoirs constructed in and shortly after 1861 were very tive, both in level and in alignment, their execution having entrusted to native clerks of the collectors' law courts.

Even under these extreme disadvantages, the works paid in 1 as much as 10½ per cent., although the water rate was inc only two years before. Of the total acreage irrigated in 18 10 919 acres were under ree quarters of which were and 168 acres in grass; e being supplied by the reser and 3421 acres by the hils. The estimated value of crops of the year was espective of the plantations, at present cousist of la

The Bandalkand Ir ks consist of five lakes and voirs in the Hamirpu. in the Jhansi, districts; they unfortunately remained control of the tax collectors, little is known of the correct ount of land irrigated by the certain amount is irrigated free of water rate, although an incr land rate is levied on it. The names of the tanks and lakes are:

	Miles of Distributaries.	Acres Irrigated.
In Jhansi.		
Kucha Bhawar ...	3½	7
Barna Sagar ...	8½	260
Kuchni	16	164
Pachwura	11	10
Total ...	39	441
In Hamirpur.		
Bejanagar, three	7	176
Desrapur, four ...	2	254

	Miles of Distributaries.
In Hamirpur.	
Thannah	5
Tikamau	1
Paswara tank ...	...
Kirat Sagar ...	¼
Maddan Sagar ...	¾
Kallian Sagar ...	¾
Bijanagar tank ...	...
Phulbagh	2
Bela Tal tank ...	...
Total ...	19¼

The former works irrigate the land of thirteen villages, the that of sixty-one; about three-quarters of the crops grown are c including rice and one-fifth sugar-cane. Some approximate fin results of these works will be found in the tabular statistics. It contemplation to increase the irrigation from these works to acres.

he Agra Irrigation Works.—These works consisted mainly of the ahpur Sikri Basin, and its channels the Khairagarh and Barkol, ich were supplied with water by the Utangant torrent. The latter s in Jaipur, flows through Bhartpur, and enters the Agra district out 7 miles east of Fattahpur Sikri. The revenue derived was not ly from the water that passed ipto the channels from the overflow the Utangan, but from the cultivation of a portion of the area of e basin itself. The irrigation from these works being very irregular, d objections having been rased against them on sanitary grounds, e works instead of being improved, were abandoned in 1865. At t time the capital outlay had amounted to £22 312, and the total ect income was £11 077, independently of increased land revenue, ich probably amounted to as much more; the yearly direct income ried between £400 and £1400, the working expenses from £600 to 200. It would appear therefore that, as also in the more recent se of the Agra canal, irrigation from which is not to be allowed thin 5 miles of Agra, there are some traditions of local magistrates d tax collectors that are opposed to irrigation.

The Rajputana Irrigation Works in Mhairwara and Ajmir consist of number of reservoirs, or tanks, having banks generally of earth, ough in many cases pitched or faced with rubble, and having masonry irs and escapes: they were made or reconstructed under the orders Colonel Dixon, the political agent, and had the beneficial effect of tling the rather troublesome population of those districts, and creasing it from 39 658 in 1835 to 130 282 in 1845; the cost on ginal works being according to old accounts only £24 111, from 835 to 1846, and resulting in an increase of annual revenue of 1 300 in addition to £9680 obtained annually till then. The llowing are data of these works according to old accounts:—

Tank.	Surface. Acres.	Contents. Cub. Yards.	Irrigation. Acres.	Tank.	Surface. Acres.	Contents. Cub. Yards.	Irrigation. Acres.
sani ...	278	5 614 400	273	Sarnagar	109	2 934 688	...
harwara	161	3 900 000	...	Tarwaja ...	218	387 200	364
abra ...	182	4 302 222	204	Rupana ...	25	524 080	36
lkankar	182	3 699 996	437	Gohana ...	94	2 684 586	250
rathu ...	167	4 701 666	...				

e extreme depths varying from 15 to 28 feet.

In 1867 these works were examined by an officer of considerable gation experience, Captain F. J. Home, R.E., and the accounts their financial results, which were then considered exaggerated, entirely readjusted: it is from his report therefore that the abstrac of financial results given in the tabular statistics has been compiled. In consequence of the number of tanks, nine varying so considerably from that for which the more recent returns are given, namely, six, it is impossible to institute a perfect comparison between the two sets of returns; but it is perfectly evident that the gross return of 47 per cent, shown by the older returns, may be generally correct. It appears also, according to other accou the total number of tanks in Mhairwara must be conside y cover a total area of 8675 acres, and irrigate 14 826 ac

In the other states of Rajp der native rulers, there have doubtless been a large numb in fact, the strong affinity of race between the Seljukian Maisur and Rajputana would lead one to believe that th been a strong similarity in condition of the two coun is still covered with tanks, and it is hence probable tha as also as much developed in this respect as its physicia nd limited rainfall allowed. In Udaipur there are still one or two magnificent lakes, and in Marwar, Jaipur, and Bhartpur, there are traces and ruins of large reservoirs, in some cases nearly obliterated by drift sand: the primary cause of the decay of these states was doubtless their proximity to the seat of government of the Mughal emperors, who plundered and devastated them; and it would at first sight appear surprising that under British suzerainty they have not recovered and reconstructed their large and numerous reservoirs of irrigation. The causes are probably these: these states do not yet possess the confidence of the British capitalist; and hence, in order to carry out extensive works, they would have to borrow from native bankers at an interest of 10 or 12 per cent., while the works under good management would probably eventually only pay 18, and in a partially developed state only 9 per cent.: in the second place, in order to design and execute the works really well, they would require the services of skilled civil engineers. On this latter point, difficulties are thrown in the way by British officialism. In former times, Englishmen and Europeans were prevented from entering into the service of native princes from fear of their using their skill in assisting in military operations and rebellion against the British Government: at present, although this fear can hardly be said to exist, the tradition still remains in the minds of the British political agents, many of whom prevent the native princes from engaging the services of *independent* Englishmen, and by persevering in this childishly weak y, put an effective bar to the development of agriculture, and quently to the material progress of native states.

e *Tanks of the Bombay Presidency* are comparatively very few, and is very little information about them available. In the district of r in the Narbada valley, is the lake of Lachma, a tank three miles cumference;—this with 105 other tanks have been restored since British occupation The Chuli tank on the Chuli ravine, and the leshwar tank on the Chapra, both in the Narbada territory, restored in 1846 by Captain Trench. In Gujrat a reservoir pro- in connection with the Tapti, intended to irrigate 194 000 acres, is carried out. In Kandeish a storage reservoir in the Girna valley, the Mukti reservoir, near Dhulia, are being constructed: the latter catchment basin of 50 square miles, which with a rainfall of 16½ es, will collect 477 millions of cubic feet, of which the tank will about 346 millions. The Hartola tank in the same district is ly completed. In Dharwar, the Maddak tank has recently been structed; and some storage works in the valley of the Yerla, a tary of the Krishna, are being made. The Ekruk tank on the a, a tributary of the Bhima, in the neighbourhood of Sholapur, was pleted in 1869, and supplied water for irrigation in 1871. The wing are the data of the original project, which was carried out . D. Campbell, Esq., C.E.—

atchment area 141 square miles, minimum annual rainfall 12 es
lood discharge of Adila river 37 000 cubic feet per second.
flood lasting five days gives 11 000 cubic feet per second.
all of Adila river 7 feet per mile, or 1 in 754.
rea of reservoir 6½ square miles, maximum depth 60 feet.
ontents of reservoir 2222 millions cubic feet = 6½ inches over chment area.
Calculated maximum velocity over waste weir 10 feet per second.
Waste weir discharge 250 × 5 × 10 = 12 500 cubic feet per ond.
Total length of dam 7200 feet, including 2730 feet masonry.
Maximum height of earthwork 72 feet, or 7 feet above flood line.
Height of masonry 3 feet above highest flood, exclusive of 3 feet of apet above.
Evaporation of 7 feet deep during eight months = 750 millions ic feet.
Unutilized residue in bottom of tank 20 millions cubic feet.

It has three canals of discharge.

i. The lowest, perennial, 28 miles long; its head is 20 feet a the level of the bottom of the tank, having a discharge of 44 cubic per second, an area irrigable from it of 25 square miles, 8 mon 912 millions cubic feet.

ii. The next for a four months' supply, 18 miles long, having discharge of 42 cubic feet per second, an area irrigable from it 21 square miles, 4 months, 435 millions cubic feet.

iii. The next for a four months' supply, 4 miles long, having discharge of 21 cubic feet ad, an area irrigable from it 10 square miles, 4 months, ns cubic feet. The discharge one 4 months' channel will eated by the mansun supply.

The duty of water for r s fixed at 96 acres per cubic f per second, and that for all ther at 150.

Acreage under command res.

The water rate for pere is 16s., and that for one sea crops 8s.

The calculated cost of t was £100 937, including 15 pe cent. for establishment; the probable gross revenue will be eventual £11 820, and the cost of maintenance £2323, at 3 per cent. on the out lay; this will yield a net revenue of £9491, or 9 per cent. on the capital expended.

The Tanks of Haidarabad are extremely numerous, the whole of the eastern portion of this state, which consists of black cotton soil, is thickly studded with them. They are all of the Madras type, similar to those of the neighbouring districts of Karnul and Ballari, and are believed to be in a very bad state of repair. There are also a few large artificial lakes, as, for instance, the Hosen Sagar near Sikandarabad, and traces of others, that at one time must have supplied a large amount of irrigation. There is unfortunately no information available as to their number or effective power, Haidarabad being an independent state extremely jealous of external interference. Latterly, however, the Nizam has engaged the services of two or three English civil engineers, and it is hence very probable that he has also commenced the repair and reconstruction of these tanks, with the view of redeveloping the irrigation of his province.

The Tanks of the Central Provinces and Berar are like those o Bombay, comparatively few and generally of small size; the Kanhan reservoir project, which involves a storage reservoir covering 41 squar s, a main canal 142 miles long, and minor channels of 400 miles the aggregate, is still not commenced. In Berar, a fertile cotton ducing province that would gain enormously from the advantages irrigation, the tanks are few, small, and in a neglected condition: was at one time imagined that any large storage projects for irrigaon in this province would be perfectly impracticable owing to the nfiguration of the country; yet in 1870, three large storage reseroirs were proposed at Donad, Balapur, and Akola, as well as several maller ones, by a civil engineer appointed by the Government of ndia. Most of these projects were then set aside by the provincial ead of the Public Works Department, a military man totally diselieving in the advantages of irrigation; it is, however, now probable hat under future more enlightened auspices, Berar may be changed nto a well irrigated and permanently prosperous province

The Tanks of the Madras Presidency are exceedingly numerous, and some of them are of immense size. They were made under the auspices of the Telingi rajahs. In the fourteen districts of Madras there are said to be 53 000 tanks, having probably 30 000 miles of embankments, and 300 000 separate masonry works, weirs, and escapes, yielding a revenue of £1 500 000, and having a capital sunk in them of 15 millions sterling; yet in 1853 not one new tank had been made by the English, while a very large proportion of them had been allowed to fall into disrepair.

The Viranam tank, a very ancient work, has an area of 35 square miles, and an embankment 12 miles long; it is still in full operation, and secures an annual revenue of £11 453.

The Chembrambakam tank in Chinglipat resembles a large natural lake, its embankment is more than three miles long, and it has six waste weirs with a total width of 676 feet of escape; it supplies 10 000 acres of rice cultivation. This tank was enlarged in 1867, at a cost of £41 000.

The Madrantakam tank at Chinglipat yielded a gross return in 1872 of £1697, and a net return of £1607 on a capital outlay, probably spent in repairs or reconstruction, of £2248.

The Kaveri-pak tank in North Arcot is also of great antiquity; it is fed from the Paler river, and has an embankment nearly four miles long, reveted with stone along its entire length; it irrigates about 7700 acres. In 1872 its banks were much damaged by an extraordinary flood, and some repairs were therefore made. There is a large number of tanks in the deltas of the large rivers of Madras, the irrigation from which is unfortunately mixed up with that from th deltaic canals in the official reports and returns.

In fact, the paucity of trustworthy statistics of the tanks Madras, on which the agricultural prosperity of so large a portion India is dependent, and on the repairs of which all capital judiciou spent seems to yield from 20 to 50 per cent., is most surprising.

The Tanks of Maisur are of native origin; they are exceeding numerous, the whole country being amply supplied with irrigation b many series or chains of then [illegible] however, owing to the con figuration of the country of s [illegible] epting in a few cases. Th are in a very deteriorated [illegible] d have suffered greatly fro silting up and want of r [illegible] d management. The larg amount of water utilized in [illegible] ur, is indicated in the table of the rivers of that provi [illegible] fortunate that the irrigate acreage due to tanks and [illegible] nseparably mixed in officia records. Maisur, althoug [illegible] elevated from 2000 to 300 feet above mean sea level, [illegible] xception of the Mulnad o rainy tracts of the Western Ghats, a small amount of rainfall, thus forcing water storage as an absolute necessity on its population; it, on the other hand, has the disadvantages of a sandy, and hence leaky soil, and comparatively steep surface slopes, the longitudinal slopes varying from 10 to 20 feet per mile in the flatter portions, and 60 to 80 in the steeper portions of the country, and more rapid transverse slopes; the former enhancing the cost of storage, the latter diminishing the breadth of irrigation from the channels of distribution. Stone is abundant, and is worked into rough forms, though too hard to be dressed for ordinary work. It is a gneiss of horizontal cleavage, which splits into sheets 3 to 24 inches thick, and 25 to 35 feet long, and is excellent for slabs and pillars, too hard to be dressed for ordinary work. For pitching, natural boulders are used, which are generally very round. Clay, on the other hand, is very rare; and lime is generally to be found only at great distances, and is hence often dispensed with in anicuts and overfalls, which are made to depend for stability on the size and position of the boulders.

According to the returns of 1853, there were 26 450 tanks in Maisur, of which 4106 were large irrigating reservoirs, 13 737 small, and 8609 unirrigating, *i.e.*, in a useless condition; giving about 1 effective tank per square mile in the gross; the area of Maisur being 27 269 square miles, of which 60 per cent. is under the tank system. In the seven *districts* of Kolar, where there are moderate conditions of rainfall, no very large reservoirs, there were 3611 tanks, of which 2950 e irrigating, giving 1·07 tanks to a square mile, and an approxi- te average quantity of wet cultivation of 10 acres to each tank. the comparatively rainless tract, comprising portions of six tricts, on which the annual rainfall varies between 10 and 20 hes, there were 1009 tanks, giving 0·31 irrigating tanks per uare mile, and 2·5 acres of wet cultivation as an average to each nk. After that time a certain amount of money was spent in pairs. In 1866, however, the Executive Engineer of the Bangalor ivision had reported that fully half the tanks under his charge were eached, in Chittaldrug 285, or one-third of the recorded number, ere out of order; in Tomkur, 530 out of 1124; in Shemugah, 2496 t of 4520; and in the Maisur Division, 705 out of 1409. Hence, it ppears, that there were in all about 1500 larger tanks requiring pair at a rate of £300 each, and 3000 smaller at £150, and that a tal outlay of £900,000 was necessary to put them in good order.

In 1872-73 as many as 249 tanks were breached. The Irrigation epartment of Maisur is now dealing with the matter gradually, by ringing the tanks up to a certain standard of repair, and then handing em over to the superintendence of the tax collectors; by these eans it is hoped that the tanks of Maisur will be economically rought into good condition.

Among the very large reservoirs requiring special notice, are the aggar Sulikerrai, on the river of that name, which has a margin of bout 40 miles, and an embankment 1000 feet long, 84 feet high, and 00 feet breadth of base; the Maddak tank on the Vedavatti, whose nbankment is 1220 feet long, and 90 feet high, having a breadth of ase of 660 feet; and the Motitalao, on a feeder of the Lokani, having embankment 117 feet high, 225 feet long, and a breadth of base f 375 feet. These are in specially favoured situations, between two ills guarding the outlets of large valleys. The proposed Mauri Cunawai d Kumbarcattai reservoirs have similar sites.

Description of an average Maisur Tank.—Length of dam ¼ to 1½ iles; 18 feet high, 12 feet top breadth, 60 feet base. Front revet- ment of rough stone, with a batter of 1 to 2, its facing 1·5 to 3 feet hick backed with the same thickness of loose rubble; sluices 1 to 3 o each tank; section of vent 2½ feet × 2 feet, length 30 to 120 feet, orm of section sometimes barrel-shaped, sometimes rectangular, they ad off from the lowest point in the tank. Inlet cistern 3 feet high, feet square, outlet cisterns the same; plug pole and gibbed stones fo

orifice ; escape weirs 1 to 4 for each tank, 30 to 300 feet wide, ma
largest stones, water front 3 to 9 feet deep ; dam stones 3 fe
4½ feet high, which when dammed give 2 feet more water ; wi
3 to 6 feet high, converging and afterwards diverging ; tail pave
sloping for a long distance or horizontal : a lower stone wall i
times placed across the tail at some distance off to intercept
the escape water, which is taken off by a channel.

WATERWORKS OF INDIAN CITIES.

ombay Waterworks, by Henry Conybeare, M.I.C.E., and — Walker, C.E.

y was the first of the Indian cities to carry out for itself ks on a modern system, and call in the aid of English civil to design and superintend their execution.

4, Mr. Henry Conybeare determined that the Vehar basin, lley of the Goper, was adequate to the collection and storage e water that would be required for Bombay for some years; s were therefore confined to the formation of one artificial d their execution entrusted to Mr. Walker as Resident in 1856. The catchment area was 3948, and was capable extended by catchwater drains to 5500 acres; the annual 124 inches, of which it was calculated that six-tenths or hes would be available, would in these cases supply 6600 million 9000 million gallons. The storage capacity allowed was 10 800 gallons; deducting from this the loss from evaporation, which hes per month for the eight dry months of the year, would to 1000 million gallons, the available supply would be 9800 As the annual rainfall on the gathering grounds greatly d the annual consumption of Bombay, it was evident that the ould continue to rise in the lake from the commencement to of the rains, or for three months, leaving only nine months' ption to be provided for. Hence, the reserve allowed in the s equal to 9800 — 3700 = 6100 million gallons, at an allowance twenty gallons per head per day for a population of 700 000 nine months, and was thus nearly equal to two years' supply.

filled up to the level of the waste weir, the maximum depth of ar lake is 80 feet; it covers an area of 1394 acres, and stands above the general level of Bombay. The three dams by which er in the lake is impounded are as respectively 84, 42, and 49 xtreme height, and 835, 555, and 936 feet in extreme length top, and they altogether contain the following quantities as earthwork, 406 066 cubic yards; puddle, 55 059; broken nder pitching, 1983 cubic yards; and pitching, 53 617 square

The top width of dam No. 1, which carries a road, is 24 feet, t of the two others 20 feet; the inner slope of all three ments is 3 to 1, the outer 2½ to 1; the embankments were d to be formed in regular layers less than 6 inches thick, , punned, and consolidated. The puddle walls are 10 feet

wide at the top, and batter 1 in 8; the trenches for foundati
excavated through the surface rock and past all surface spri
the solid basalt below; the slopes and tops of the dams were
with 12 inches of stone pitching over 12 inches of broken ston

The waste weir is 358 feet long, and has a top width of 20 fe
with ashlar. The water is drawn from the reservoir through
provided with four inlets, at vertical intervals of 16 feet, havin
meter of 41 inches, and provided with conical plug seats faced w
metal—the plugs being suspended from a balcony, and work
cranes at the top of the e inlet in use is surmounte
wrought-iron straining d with No. 30 guage coppe
and fixed to a conical nto the inlet orifice in th
manner as the plugs, a able of being raised or lowe
pleasure: the strainer of 54 square feet. The stra
so affixed to the cage its being changed in ten m
from a boat, and a p or the cage. At the bottom
inlet well, and exactly nce to the main, is another c
seat, into which a si age, having a surface of 90 s
feet of No. 40 gauge copper-wire gauze is inserted. The objects
arrangement were to utilize the whole head of water, includin
due to the depth of the lake, which would have been lost had the
been strained at the outside foot of the dam; and to avoid the
heavy sluice-valves, in positions in which it would be difficult to
them. Without this, the utmost head obtainable would have be
sufficient for a distribution by gravitation alone. No filtration ar
ments nor sludge-pipe were considered necessary.

The suppy main traversing the dam is 41" interior diameter, a
metal 1¾ inches thick: it is laid in a level trench excavated in th
and filled with concrete: the portion traversing the puddle tre
supported on ashlar set in cement, puddled to a depth of 6 inche
then arched over with four rings of brick in cement; two tea
washers being affixed transversely on the pipes to prevent any
from passing between the pipes and the puddle. At the sluice-
situated at the outside foot of the dam, the large main, 41 inc
diameter, bifurcates into two mains, each 32 inches, which co
for a distance of nearly 14 miles to Bombay. The supply is distr
through the town by branch and street mains in the usual way
hydrants are self-closing, and of a design that admits of their c
either with or against the water pressure, the counterweights
adjusted to the resistances at the various levels of the town: the
valves, 32" diameter, are so constructed as to be capable of being

ned under the severest pressure, with a very trifling exertion of the smaller valves are on Underhay's system, which admits of moval of the valve seat and valve, without disturbing the laying y portion of the mains. The water is delivered under a pressure m 165 to 180 feet. The actual delivery of water commenced in , 1860. The original estimate of these works was £250 000; cost, including interest, was £655 000. The result was a supply ellent water to Bombay of 8000 instead of 9800 million gallons bringing in an annual revenue of £38 000. At present, in 1873, the population has increased to 800 000, the supply per head ts to only 10 gallons daily, and an additional supply is required. s projects, having this object in view, have been proposed by Mr. ll Aitken, Captain Hector Tulloch, and Mr. Rienzi Walton, C.E., ipal Engineers, and a very large amount of time has been spent assing them.

The Madras Waterworks.

for the Water Supply of Madras and Irrigation near it, by W. Fraser, C.E., Executive Engineer.

original estimate of the works was as follows:—

A dam across the Cortelliar stream	£3 071
A channel with head and other sluices, bridges, and other requisite works, for 8½ miles, from the dam to Cholaveram tank	12 206
The enlargement of this tank by raising its embankments 18 feet	15 239
A channel 2¾ miles from the Cholaveram to the Red Hill tank, with sluices, bridges, and other works	6 596
The enlargement of this tank by raising its embankments 15 feet	11 793
A channel from Red Hill tank to the Spur tank in Madras, with sluices, bridges, and other works ...	2 803
ndries, compensation, superintendence	13 348
	£63 693

consequence of alteration of design and increase of rates the quent revised estimate amounted to £104 264.

e dam as erected was 469 feet long, and 6½ feet high at crest, to 8571 acres of rice, at 7000 per acre, yielding £6000, at 14s. cre, and 33 millions for water supply. Assuming that the ation of Madras will increase from 170 000 to 500 000, and will e a supply of 20 gallons per head daily, their wants will not d 22 million cubic yards per annum. The distribution of the supply from the Spur tank forms a separate municipal under- g: the municipality of Madras agreeing to pay 1 rupee per 1000 yards of water taken from it.

e original rates of work per cubic yard were—earthwork of all 2¼ to 4 annas; puddling, 6 to 8 annas; revetment, 8 annas; work complete, 3 rupees to 3 rupees 4 annas; thus, quarrying quaring, 1 rupee 8 annas; cartage, 2¾ miles, 1 rupee; building, nas. These rates were afterwards increased.

e capital outlay up to the end of 1871-72 was £104 772, but further sums were spent during 1872-73; from which it would that the Madras waterworks are now nearly in perfect working ; the income and cost of maintenance up to 1872 73, was £222 £2911 respectively; and during 1872-73, £1516 and £667.

The Calcutta Waterworks.

gned by W. Clark, C.E., in 1865, carried out with alterations by — Smith, C E. The intended daily supply, 6 million gallons.

eral Design.—The water is drawn from the river Hughli at Pultah, iles from Calcutta, through an iron suction pipe protected from urrent by an open iron jetty, the suction boxes, 36 inches, being ed with an iron sheet perforated with one-inch holes. The first es are situated at Pultah, close to the river; they are three in ber, high pressure, double acting, expansive, condensing, of 30 ., nominal 5 feet stroke and 30 feet lift, and pump twice a day g low water, for five hours each time, into the settling tanks to them. The settling tanks are six in number, each being 200 0 feet, are used and cleaned in regular rotation: it takes one h to clean one, the deposit of mud being very large, even as much cubic inch to the cubic foot, or 1 part in 728 in bulk when dried. wever this has to be removed from the settling tanks in the fluid of soft mud about three or four times daily, the above proportion lk amounts to or from 4000 to 5000 cubic feet of mud daily from lion gallons of water. For cleansing the bottoms of the settling are arranged in a series of corrugations 48′ 6″ wide; on each of dges a drain 4′ wide by 1′ 3″ deep is formed, into which the water ah, the course being generally alongside of a high road. The ible fall from Pultah to Calcutta, a distance of 17 miles, is about feet. This covered reservoir, intended for storage in emergency, is 0 × 2[illegible]0 × 20 feet, of which 16 feet is available for storage, holding thus 8 million gallons. The bottom consists of a series of verts 15 feet span, and two rings thick, turned on a floor of 6 inches concrete covered with a layer of asphalte. The outer walls are 2′ 6″ ck, plastered with cement.

From this reservoir engines, three in number, and similar to those Pultah, any two being able to carry on the work, pump during daytime the supply required for twenty-four hours for the northern ision of Calcutta into the trunk-mains, and during the night-time required for the southern division of Calcutta into a covered ervoir at Wellington Square: for both these purposes the engines the water from the bottom of the reservoir to a height of 50 feet ve the bottom.

Distribution.—The distribution is effected from the store reservoir Tallah in two divisions. 1st. A 30-inch inlet-main from the works Tallah to the canal aqueduct, thence continued up to the Circular Road, 1408 yards. 2nd. A 24-inch main from the junction of Circular Road and Cornwallis Street to Wellington Square 4864 yards long. This pipe serves during the daytime as a main to supply the northern division of the town at a low pressure of 50 feet head, and at night to fill the tank at Wellington Square; whence the supply of the southern division is carried on by engines under high pressure.

The engines at Wellington Square are three in number, and of similar principle to those at Pultah and Tallah, but are of 75 H. P. nominal; any two will do the necessary work, the power being that necessary to distribute the full daily supply in six hours from the level of the bottom of the reservoir to a height of 100 feet above the surface, or a total lift of 120 feet. The work actually done by two of these engines, in ordinary practice, is to raise 132 gallons at each stroke, at a speed of 20 revolutions per minute, or in thirteen hours with three tons of fuel, to raise 3½ million gallons under a lift pressure of 60 feet.

For the low pressure division there is also an auxiliary 18-inch main, 1345 yards long, and two 12-inch mains, both together 2980 yards long. For the high pressure division, the auxiliary and lateral trunk-mains are—one 24-inch main, 220 yards long; three 18-inch mains amounting in length to 3840 yards; and ten 12-inch amounting in length to 6965 yards; exclusive of two trunk-mains 12-inch and

9-inch c and 1465 yards long respectively. The whole le of the main pipes in yards being thus :—

	30"	24"	18"	12"	9"	Valv
Southern Division	..	220	3840	8585	1465	14
Northern Division	1408	4864	1344	2980	...	10
Total	1408	5084	5184	11565	1465	24

These mains have also district service mains in loops or sectio closable by valves as fol In the low pressure division they a 13 in number, in the h division 26, consisting of following lengths in ya

	9"	4"	3"	Valves.
Low pressure	1830	5 414	1912	24
High pressure	2214	17 212	8336	48

The water-pipes are along the streets on the si opposite to that of the y are in 9 feet lengths, and the weights usually ad

The total length in yards of the mains are as follows :—

	Trunk mains.	Loop mains.	Total.
Low pressure Division	10 596	24 064	34 660
High pressure Division	14 110	42 964	57 074
Totals	24 706	67 028	91 734

or about 52 miles.

The inclinations adopted are as follows :—From Pultah to Tallah 1 in 5500 ; sludge culvert, 1 in 500 ; river water culvert, 1 in 1600 clear water culvert, 1 in 1000.

The loop system being adopted in all future extensions or ne district mains, dead ends are altogether avoided ; so that on openin the valves connecting these mains with the trunk-main, a free circula tion must take place throughout ; the loops cannot be connecte together, but additional pipes can be inserted into any of these loo to obtain an extended distribution. The pipes allowed are fully ab to distribute 12 million gallons daily, or double the amount at prese required. It is intended to keep the pipes constantly full under pre sure, so as to obviate any necessity for cisterns.

Besides the above supply for Calcutta, the works will give eventual a supply of 120 000 gallons daily to the cantonment of Barrackp involving an elevated tank 50 feet high, 4660 yards of 9-inch ma pe, and a supply of 60 000 gallons to the cantonment of Dam-Dam, nder a pressure of 50 feet through 6600 yards of 6-inch pipe.

The total cost of the water delivered in Calcutta, half at 50, and alf at 100 feet pressure, is estimated at about 30 000 gallons for £1.

The delivery of the main supply commenced in 1869.

The estimated prime cost was—

Price and rent of land taken for the works ...	£11 082
Machinery and Works, engines, filters, reservoirs, pipe to Tallah	377 838
Trunk and district mains, valves and hydrants, after deducting for valve of some received ...	106 676
Total...	495 596
Engineering and contingencies 15 per cent. ...	75 000
Supply to Barrackpur and Dam-Dam	10 500
	£581 096

The annual expenses are estimated at £75 964, inclusive of £57 060 for repayment of loan, at 10 per cent on cost of works.

The Ambajhari Reservoir, constructed by A. Binnie, M.I.C.E.

The name of the projector of this scheme, which is an enlargement of a native tank, is not mentioned in the official records: it was chosen from among other projects for the supply of Nagpur, by Mr. Binnie, in 1869, and laid before Government in the two following forms:—

Project No. 1.—*Water Supply of Nagpur.*—Population, 84 000; catchment area, 6·6 square miles, bare and basaltic, having an annual rainfall 40·73 inches, mansun rainfall 37·52 inches. Proportion run off in an average mansun ·43, minimum ·268, maximum ·6.

The evaporation is based on Conybeare's measurements at Vehar, Bombay, which give 2·5 feet in eight months of dry season, or ⅛ inch daily, hence allowance is made for 3·5 feet in eight months as a maximum for Nagpur. The rate of silting determined from observation to be 2·5 feet in 80 or 90 years = ·375 inches annually. Supply allowed 7 gallons per head daily, and as this is all wanted nearly at one time, the pipes are made to deliver 15 gallons per head daily. There is no filtering arrangement, but strainers of copper-wire gauze are used, being fixed in wooden frames in the inlet tower. The siphon is 2·5′ in diameter, length 185, rise 15, fall of 2′ to overcome friction: air pipe 3″ diameter. The siphon joint

are turned and bored, flanges packed with wood, bolted and fasten with hoop iron, bolts and washers. The maximum head is 78 fe or 34 lbs. per square inch, hence the pipes are tested to 130 lbs. per square inch. The formula used for the discharge of pipes i Young's Eytelwein, $v = 50 \sqrt{\left(\frac{dh}{l + 50\,d}\right)}$, There are scouring valves at low points. The embankment is in layers 12 inches thick, inclining inwards 1 in 6, retentive clayey material alone used, its surfaces of hard material, covered with 12″ of rough hand pitching, its
slopes are outer 1¾ to 1, in s foundation is stepped and
benched. The escape wei rubble, its sill of angle-iron
3 × 3 × ½ welded and bo l. The waste watercourse is
18′, broad at bottom with a. The main pipe is carried on
walls of rubble, or in a bed feet thick, stepped into the
embankment; in the valve id in concrete. Pipes above
13 inches diameter to have caulked with spun yarn, and
lead driven in with caulki of less than 13 inches turned
and bored, fixed with Ron All pipes to be tested under
pressure by hammer 7 lbs. weq — gus Smith's process applied
to all pipes inside and out after fitting. Distributing pipes to bear on solid ground, in trenches 4 feet to 2½ feet deep, filled and rammed.

The puddle wall in the centre of the dam is 5 feet wide on the top, and 10 below, and 30′ high, made in layers of 8 inches.

Project No. 2, combining Irrigation with Town Supply.—Siphon as in last project; irrigation duty of water, 200 acres to 1 cubic foot per second; acreage 1121, for eight months excluding waste land = 116 225 280 cubic feet in all, including 747 acres for twelve months; distribution effected by a large irrigation pipe with wide joints, giving 7·93 cubic feet per second to start with, and decreasing in diameter so as to give only 2·32 cubic feet per second for water supply at the city 5 miles off; the intermediate points of discharge for irrigation regulating the discharge and diameter of the pipe between them: this arrangement allows 7·93 — 2·32 = 5·61 cubic feet per second for
21 acres of irrigation, and prevents an excessive supply from being
ten in the city, as it might be in an open channel. The discharges
d hence the sizes of the small irrigation outlet pipes are calculated
s if they were independent up to the reservoir; sluice cocks are provided at the branch outlets. A gauging and regulating apparatus, worked by a table of discharges calculated for every ·01 foot of rise for submerged orifices and weir, controls the whole supply.

The details of the above projects were drawn up in 1869, the former g sanctioned in April 1870, and the contemplated irrigation being rred. The estimates amounted to £32 535; the reservoir was ned in October 1872, but the distribution was not carried out by time. The reservoir has a top surface of 370 acres, and a age of 257·5 million cubic feet, of which 240 millions, or 1500 lion gallons, are available.

he cost of excavating the puddle trench, including pumping, was 68, at the rate of 1s. per cubic yard; the cost of puddle, £6659, 4s. per cubic yard; the cost of embankment, in 1 foot layers med and watered, was £4277, at 5¾d. per cubic yard; the es for pitching were from 5s. to 10s., and for turfing, 2s. per superficial feet; the total cost of the outlet, including straining-wer, foot-bridge, well and valve house, was £2893, and that of the ape weir, £821; the rates for ashlar, basalt, rubble, and concrete ng from 27s to 54s., from 10s. to 16s., and 8s. per cubic yard.

The distribution source is a public one, the water standards being ced 100 yards apart along the streets. The main pipe was 4 miles g and 1·1 feet in diameter, and the distribution pipes 10 500 yards g and 1 foot in diameter; the pipes were delivered in Bombay at 5s. per ton, and in Nagpur, at £11 14s. The works were completed thin the estimate, and a supply of 15 gallons daily per head can be intained in years of extreme drought.

e *Akola Project for Irrigation and Water Supply, by L. D'A. Jackson, A.I.C.E., Executive Engineer for Irrigation in Berar.*

The proposed works consist of—

I. A reservoir formed on the Morna river by a masonry dam and earthen embankments east and west of it.

II. An irrigation channel 5 miles to the first watershed, and 3 more to the third watershed to the east of the river, and irrigation channels 15 miles to the west of the river.

III. Filter beds, drinking and bathing basins, with a fountain at the town gate of Akola, with pipes to it 1½ miles in length.

1. *Masonry dam* 625 feet long, extreme height 36 feet; area of tion of superstructure down to 30 feet $\cdot 3H^2$, and of foundation low that $21h$; strengthened by buttresses 50 feet apart from centre centre; the wing walls rise to 8 feet above the sill level and revet embankments, which are 8 feet wide at top, slopes 2 to 1 and 3 to 1 d have a section $10\cdot 5\ H$; length of eastern wing 2751, wester 57 feet.

Reservoir, extreme length and breadth about 2¼ miles, area of wat
spread 2500 acres: of which 1000 are under cultivation, and on whi
there are only a few small huts.

Contents available for perennial irrigation, cubic feet		411 055 831
Available for town supply	,, ...	58 427 360
Waste or standing water	,, ...	8 843 139
Total contents	,, ...	478 326 330

Beside this, there will be available for mansun irrigation in season
of extreme drought at le
es the above total from the
perennial flow of the river

2. *Channel.*—Section 45
slope 1 in 3000, discharge 100
cubic feet per second belov
und level in section. In east-
ern channel 8 super passa
aving section of 60 square feet
and discharging 150 cubic
nd; 8 road crossings; 2 under
passages through embank
feet pipes enclosed in masonry
culverts. In western cha
assages, 12 road crossings, and
2 under passages. The small trenches of distribution to be made by the landowners, aided, if necessary, by loan.

3. *Town supply.*—Pipes 4 inches in diameter, having a fall of 1 in 500, and discharging ·25 cubic feet per second. Beds and basins excavated in rock, with walling above ground. Filter bed and bathing basin each 50 feet square and 10 feet deep. Drinking basin octagonal having the length of each side 40 feet, and having a jet in the centre, the water for which will be purified by a filter on the ascending principle passing through perforated walling and tiles, then large and small pebbles, sand, and magnetic carbide.

Data.—Catchment area 220 square miles, minimum downpour 12 inches of which 6 inches run off, give 3066 million cubic feet in a year of drought, and fill the reservoir six times. The extreme flood discharge over the weir sill, using a local coefficient of 12 for the formula $Q = 12 \times 100\,(N)^{\frac{3}{4}}, = 67\,200$ cubic feet per second; and assuming a flood velocity of 13 feet per second, this gives a flood section of 5170 square feet. The waterway allowed is $8 \times 125 = 5000$ square feet; the measured flood sections are in support of this.

Land under water command on the east bank 45 square miles, west 30 square miles; total 75, all fertile; the perennial supply for irrigation during the eight dry months is 410 million cubic feet, or 19·5 cubic feet per second, which at a duty of 200 acres will irrigate 3900 acres. The mansun irrigation supply for four wet months exceeds any demand that is likely to occur; the probable maximum acreage for this will be about half the irrigable area, or 20 square miles on one bank and 15 on the other, being in all 35 square miles or 22 400 acres; the channel of supply is designed to carry sufficient to irrigate the total area of 75 square miles.

Cost of Works and extension on the west bank	£31 301
Compensation and Road diversion	1 000
Establishment and contingencies 20 per cent.	6 869
	£39 170

Probable return, when the works are fully developed:—

Perennial, *i.e.*, 8 months, 3900 acres at 14s ...	£2 730
Mansun, *i.e.*, 4 months, 22 400 acres at 4s. ...	4 480
	7 210
Collection, repairs, establishment, 8 per cent. ...	577
Result, a net return on capital £40 000 of 16½ per cent.	£6 633

Or, deducting capital spent in town supply, a result of 19 per cent. on the outlay on the capital spent in irrigation, independently of the water rate charged to the town.

The classification of water rates for various crops is that adopted on the Bari Doab Canal, but the rates themselves are doubled, as the cost of labour in Berar is double that in the Bari Doab. Hence the rates assumed for Berar are,—1st class, Sugar-cane, £1 4s.; 2nd, Rice and garden produce, 19s.; 3rd, All ordinary field crops, not elsewhere mentioned, 10s.; 4th, All millets, pulses, and grass crops, 6s.; 5th, A single watering, 3s. These may be expected to yield mean rates of 14s. and 4s. at the least, as it is most probable that sugar-cane will be extensively grown; all the sugar in Berar being now imported.

IRRIGATED CROPS, WATERINGS, AND WATER RA

The Watering of Crops in Spain.

The following data of Mr. George Higgin, C.E., in 1 cate the amount of water required for crops in the irri tricts, where the annual rainfall excepting of Granada, is 22 inches only.

Actual water duty on old works.	Per sec. per hect. Litres.	Per sec. per acre. C. feet.
In Valencia, from the Jucar, rice	2·00	·0282
In Valencia, from the Turia, old	·86	·0121
In Gandia, type of old	·80	·0113
In Murcia and Orihuela, old	·74	·0104
In Granada, old...	·29	·0041
Eslà and Henares, new	·45	·0064
Lowest duty in Spain generally, new	·50	·0071

The practice of watering usual in Valencia is, for lu watering in 8 or 10 days; for maize, beans and hemp, one in for potatoes, one in 21 days; for cereals, one in 30 days; th amount given at one watering in ordinary soil is 500 cubic hectare (7060 cubic feet per acre), and the fullest ever gi (9884).

The following data of Mr. Roberts, C.E., in 1867, are s support of the above.

Average actual water duty in various provinces.	Per sec. per hect. Litres.	Water duty authorized for various canals.
In Valencia	·25	De las Cinco Villas
In Rioja (low clayey)	·20	De Tamarite
In Murcia, Alicante, Aragon, and Cataluña	1·00	Del Henares Del Esla
Cereals & grass generally	·25	Del Tajo
Huertas or gardens	·75	Del Ebro
All other lands	·50	De Isabella Segunda
In extremely dry seasons	1·00	

The practice of watering is—for cereals, &c., 4 to 6 waterin for meadows 8, and for gardens 20; each watering being i 2 inches deep, which = 550 cubic metres per hectare, and ceeding 2¾ inches, or 7 centimetres, which = 700 cubic hectare. The average number of waterings in a year give in Valencia is 12.

The Watering of Crops in France.

rom data given with reference to the Marseilles canal, in a paper d by Mr. George Rennie before the Institution of Civil Engineers 855, it seems that in Dauphiné, only one watering per week, of a th of 3 centimetres (1·18 inches), is given on heavy lands; but on t soil, and with the object of making up for losses by filtration, the pth allowed is 10 centimetres (3·94 inches); in lower Languedoc d Rousillon, the same practice of irrigating once a week, but with epth of 5 centimetres on heavy, and 10 on light land; in the ntale of Provence the same as in Rousillon for field crops, but a ater quantity for garden crops. There are, however, localities in nguedoc and Provence, where this system is practised only during e or two months, or for two or three times in the year.

The irrigation furnished by the canal of St. Julian, on the Durance, 360 hectares (889 acres), at Cavaillon, Vaucluse, was 538 272 cubic tres a week, giving a calculated depth of watering each week of centimetres over that area; and this is in support of an average th of supply actually utilized of 10 centimetres once a week.

rom data given by De Cossigny in the "Notions Elementaires sur Irrigations, 1874," the watering season, in the south of France, from the 1st of April to the 1st of October; on ordinary land in ovence the depth of watering usually given is 8 to 10 centimetres, this is supplied once in ten to twelve days during the six months; amounts to a total quantity of 15 552 cubic metres = 1 litre per cond per hectare, as a continuous supply: garden crops require tering once in five days, and require a supply of 2 to 3 litres per ond. The extreme limits are ¼ litre as a minimum, according to Pareto, and 4 litres as a maximum, according to M. Mangon. For rious soils the same amount of water is given at each watering, but waterings are more or less frequent, varying from once in five ys for soil four-fifths sand, to once in fifteen days for soil one-fifth nd. Summer meadows require a depth of from 5 to 10 centimetres each watering, or a continuous supply of from ¼ to 4 litres per cond per hectare, or an average of from 1 to 2 litres per second; though they can, according to M. Mangon, utilize and profit from as ch as from 34 to 50 litres per second. For winter meadows the nimum supply advisable, according to M. Zeller, is a depth of 13 ntimetres at each watering, or a volume of 1300 cubic metres in enty-four hours, which is 15 litres per second per hectare; th aximum which they can utilise is 1700 litres per second pe hectare, ir se of the Prairie Habsaurupt; and an averag allowance is from 0 to 50 litres per second per hectare. Rice crop are considered to require a permanent depth of from 15 to centimetres on them, in some cases as much 40 centimetres, a continuous supply of 1½ to 2 litres per second as a minimum permanent stagnation of the water is considered very unhealth Most crops in the South of France, more especially fodder and ro crops, require or greatly profit from irrigation. Oleaginous plan and arborescent cultivation quire it. Vines are floode to a standing depth of 10 nd kept thus for a month i winter; this destroys the nd renders the vines mo fruitful in the following s

The W in Italy.

According to old data, th duty in Piedmont and Lombardy was from 60 to 80 f. p. sec., in some cases from 90 to 100, and rarely 110. lected by the author in Italy in 1872, the duty under o ances is considered to rang between 80 and 110 on the works. The occasion of the execution of the Lago-Maggiore project by Signori Villoresi and Meraviglia, led to a re-examination of the subject; and data were furnished by Signor Cantoni, Director of the School of Agriculture at Milan, and by a special committee of engineers. The principle adopted is that of the French, namely, that the amount of each watering to all land should be identical, and that the number of waterings alone should vary with the soil and the crop.

The following are means of results determined by De Regis, Cantoni, and the committee. The amount necessary for meadow land at each watering is 15 045 cubic feet, of which 9160 is utilized, and 5885 is absorbed; the number of waterings given varies from one in 7 to one in 10 days, thus giving a duty of from 40 to 57 acres per cub. f. p. sec.; sandy lands requiring ·025 cub. f. p. sec. per acre, and clayey lands ·017. The amount necessary for arable land at each watering is 18 173 cubic et, of which 9697 is utilized, and 8476 is expended: the number of aterings given varies from one in 14 to one in 20 days, thus giving duty of from 66 to 100 acres per cub. f. p. sec.; sandy lands requiring 015, and clayey lands ·010 cub. f. p. sec. per acre. The average of the irrigable land under the Lago-Maggiore project, amounting to 193 690 acres, requires a supply of ·012 cub. f. p. sec. per acre throughout the year, or a duty of 90 acres; the maximum duty for clayey arable land being fixed at 110 acres.

rigated Crops of the Panjab and the value of an acre of produce in 1872.

The Western Jamna Canal, 1872.		Produce per acre.	Market value in 1872.	
		lbs.	£	s.
r-cane—Saccharum officinarum	Annual	2000	7	14
en produce ...	„	...	8	0
—Oryza sativa	...	1920	3	6
on —Gossypium herbaceum	...	720	3	12
p—Crotalaria juncea	...	200	1	5
go—Indigofera tinctoria	...	20	1	3
ower—Carthamus tinctorius	...	120	3	0
meric—Curcuma longa	...	...	...	
Sesamum—Orientale	...	160	0	16
a	...	640	1	10
tard—Saru Sinapis campestris	...	400	1	0
seed—Linum usitatissimum	...	120	0	12
ernuts—Teopa bispinosa	...	6400	8	0
acco—Nicotiana tabacum	...	2800	0	18
py—Papaver somniferum	...	240	3	0
nia—Coriander	...	5600	2	6
aun	...	5600	2	6
wen—Ptychotis	...	400	2	0
hi—Trigonella fœnugrœcum	...	400	2	0
ar—Holcus orghum	...	1680	2	14
gni—Italian millet ...	...	1600	2	12
ra—Penicellaria spicata	...	1520	2	10
na—Panicum miliaceum	...	1520	2	8
e—Zea mays	...	1600	2	10
eat—Triticum vulgare	...	1520	3	4
ley—Hordeum cœteste	...	1120	1	10
s—Avena sativa	...	1200	2	8
m—Cicer arietinum	...	1440	2	5
ur—Ervum lens	...	400	0	12
d—Delichos pilosus	...	1440	2	16
ng—Phaseolus mungo	...	1440	1	16
th—Phaseolus aconitifolius ...	...	1440	1	16
erne—Sinji Medicago sativa	...	3200	2	0
ntry grass	...	4800	0	15
rri—Holcus sorghum	...	3200	0	8

Seasons of Crops on the Indus Inundation Canal, Derajat, 1872.

	Usual time of		Earliest date of Watering.	Latest date of Watering.	Remarks.
	Sowing.	Reaping.			
Cotton	1st to 15th June	Oct. and Nov.	1st to 15th May	Up to date of reaping in Oct. and Nov.	Cannot be sown earlier on account of hot winds.
Rice	May	15th to 30th Aug.	1st to [illegible]	[illegible]	Sown in March, watered from wells, and transplanted in May.
Indigo	1st to 15th May	1st year August, 2nd year 1st to 15th Sept.	1st yea [illegible] 2nd ye [illegible]		Indigo for seed gets a watering in September.
Jowar	June	15th Sept.	15th to 31st May	15th to 30th Aug.	These crops require at least one watering before ploughing, and two after sowing.
Bajra.................	July	15th to 30th Sept.	1st to 15th April	1st to 15th Sept.	
Barley .... Gram........... Wheat	20th Sept. to 31st Oct.	20th Mar. to 15th April.	July	September	Require 1, 2, or 3 waterings before sowing, and sometimes one after sowing, but not often

N. B.—The inundation of the Indus [illegible] in May.

		Time of Sowing.	Time of Reaping.	Earliest date of Watering.	Latest date of Watering
The Western Jam[illegible] Canal.					
Kharif	Sugar-cane	Feb. to April	Nov. to Feb.	1 March	28 Feb.
	Cotton...............	March to June	Sept. to Dec.	1 March	31 March
	Rice..................	May to July	Sept. to Oct.	1 May	31 Oct.
	Jowar	June to Aug.	Sept. to Oct.	1 June	15 Sept,
	Maize	May to Aug.	October	1 June	15 Sept.
Rabbi...	Wheat............	Oct. to Dec.	April to May	1 October	31 March
	Barley				
	Gram				
Bari Doab Canal.					
Kharif	Cotton	25 April 25 June	15 Sept. 13 Dec.	11 April 25 April	25 Sept. 20 Oct.
	Rice	26 May 26 June	15 Sept. 4 Oct.	13 May 30 June	28 Aug. 15 Sept.
	Sugar-cane	8 Feb. 11 April	5 Oct. 8 Feb.	11 Jan. 9 Feb.	28 Aug. 15 Sept.
	Indigo................	25 April 15 Sept.	25 June 15 Nov.	12 June 28 July	21 Aug. 21 Sept.
	Jowar	12 May 26 June	15 Sept. 20 Oct.	12 May 11 June	27 Aug. 21 Sept.
	Maize	do.	do.	do.	do.
Rabbi...	Barley..............	15 Oct. 27 Dec.	11 April 5 May	15 Sept. 14 Oct.	19 March
	Gram	15 Oct. 29 Oct.	31 Mar. 16 April	27 Aug. 29 Sept.	not after sowing
	Wheat...............	15 Oct. 29 Oct.	31 Mar. 16 April	15 Sept. 14 Oct.	24 March

Experiments in Watering Crops of Wheat and Rice on the Bar Doab Canal, by E. C. Palmer, C.E., in 1871.

The average of the experiments made and tabulated show that a average depth of 0·24 feet on the whole surface, represents a thorou watering of the average soil of the district under consideration, an for sandy soils 0·31 feet, and the amount of water necessary for a average watering of one acre, is 0·24 × 43 560 = 10 454 cubic feet.

Wheat in a dry season r [illegible] waterings; the first, for pre paring the land for ploughi [illegible] 0 cubic feet, and four for th standing crop of 8000 cubic [illegible] 42 500 cubic feet in all nece sary for each acre of wheat [illegible]

Rice requires ten flood [illegible] mount of water necessary fo each flooding is the am [illegible] ary to saturate the soil, the average of which, given [illegible] 0·24 feet, together with 0·5 feet of standing water: o [illegible] 75 feet in depth over an acr represents the quantity o [illegible], or 0·75 × 43 560 = 32 67 cubic feet; and the quantity necessary for a crop of rice is, therefore, 326 700 cubic feet.

The land under consideration principally consisted of holdings on an average of 52 acres, requiring 22 acres of Kharif, and 30 of Rabbi irrigation; for such a farm an irrigating outlet or pipe 0·4 feet in diameter, working under a head of 0·4 feet, was found sufficient; the discharge being 0·3323 cubic feet per second, and allowing the farmer eight days to prepare his 22 acres of Kharif ploughing, and eleven days for the 30 acres of Rabbi ploughing. As the best season for this purpose lasts about six weeks, and the outlets are allowed to flow for eight days in the month at the utmost, this arrangement allows twelve days of constant flow during that season; and thus a single pipe, irrigating only 2·7 acres per day of twenty-four hours for ploughing, or 5·4 acres of standing crops, is sufficient for all the purposes required in keeping up the irrigation of a holding of 52 acres.

These data are apparently in support of the amount mentioned in official returns as the average supply per acre given on the Bari Doab Canal, 44 000 cubic feet; the latter probably including also single waterings over a certain amount of acreage.

[redacted]

The Canal Plantations of the Panjab.

Western Jamna Canal.

	Number in 1872.
r Babul—Acacia arabica	394 718
ı—Dalbergia sissu	119 611
Mulberry—Morus alba	72 526
'edrela tuna	33 789
–Sizygium jambolanum	17 214
—Melia azedarach	16 764
Acacia speciosa	16 870
-Ficus cunia	11 755
Acacia leucophlœa	7 205
Azadarachta Indica	7 152
Bambusa stricta	4 911
Mangifera Indica	3 774
China—Morus tatarica	2 130
Ficus religiosa	2 004
ıneous of 80 descriptions	...
Total of all sorts	809 797

Bari Doab Canal.

	Number in 1872.
n—Dalbergia sissu	451 566
-Acacia arabica	173 124
...	71 710
ry	54 458
Acacia speciosa	47 292
Cedrela tuna	31 853
...	16 735
Prosopis spicigera	11 551
a—Ficus caricoides	9 760
ı	6 178
—Prunus padus	4 887
—Melia sempervirens	5 966
Dodonnœa burmaniana	4 850
Zizyphus flexuosa	4 689
—Bombax heptaphyllum	8 013
ıneous trees of 83 descriptions	...
Total of all sorts	955 567

The Crops of Orissa and their Waterings.

The Late Crops, watered between June 1 and December 1 :—

	On ground from		On ground fro
1. **Sarud** rice ...	April to Feb.	3. Laghu rice ...	May to Nov.
2. **Biyali** rice ...	May to Oct.		

The both Season Crops, requiring perennial watering :—

	On ground from		On ground fro
1. **Sugar-cane** ...	April to Mar.	3. Yams	May to Feb.
2. **Turmeric** and ginger	June to [illegible]	4. Brinjal	June to Jan.
		an and plantain	Whole year.

The Early Crops watered [illegible] ember 1 and June 1 :—

	On groun		On ground fro
1. **Dalua** rice ...	Feb. to [illegible]	Tobacco	Nov. to Apr.
*2. **Wheat**	Nov. to [illegible]	Coriander ...	Oct. to Feb.
*3. **Barley**	,,	Onions and garlic	Nov. to Jan.
*4. **Gram** and peas	,,		
5. **Achua** cotton	Nov. to [illegible]	Achua castor oil	Nov. to Feb.

The Dry Crops not requiring [illegible] are :—

Late Crops.

1. Mandia.
2. Biri pulse.
3. Black kulthi.
4. Black mug.
5. Jute and hemp.
6. Haldiya cotton.
7. Haldiya castor oil.

Early Crops.

1. White kulthi.
2. White mug.
3. Harar chaitra.
4. Mustard.
5. Linseed.

Both Season Crops.

1. Harar nali.
*2. Til.
Pulses generally.

N.B.—The crops marked * are rarely cultivated.

The usual rotation of the dry crops is, 1st year, Biyali rice (which, like Laghu rice, can be grown without irrigation), followed by pulses kulthi, mug, linseed, or mustard ; 2nd year, cotton, turmeric, ginger, or sugar ; 3rd year, fallow.

The country cotton is an annual ; of oil seeds, castor-oil is the only one that profits from irrigation ; pulses and linseed suffer from rain ; ginger and turmeric require only one or two waterings ; sugar-cane is sometimes planted as early as February and cut in November. There is a coarse species of rice grown in swampy tracts called boro dhan. The yield of Sarud rice, the staple crop, is said to be doubled by irrigation, and amounts to 10 cwt. per acre.

ity of Water required for the Irrigation of Rice in Orissa, from the Experiments of Mr. James Kimber, C.E.

Balagurriah Plot of 54·3 acres was irrigated by means of a 1 foot square, and a field channel 700 feet long therefrom. The iments were made in the year 1872, which had a total rainfall g the irrigating season of 53 inches. From the 7th to 14th , 1872, the water ran with ·5 foot depth in channel, and a head foot, the discharge for those seven days being 965 584 cubic feet 58 cubic feet per second; gauge readings being made four times y on each side of the field sluices. The readings reduced and ed, were averaged to give a mean daily head; from this, the nt of opening, and the number of hours open, the daily dis- ge was calculated. The total results were thus:—

amount of water given	2 885 006	cub. ft.
irrigated	2 368 028	sq. ft.
ount of water represented vertically	1·213	feet.
ber of hours irrigating	674	hours.
during actual irrigation of 1 cub. ft. per sec.	46	acres.
ctual duty on the area of 1·19 cub. ft per sec.	54·3	acres.

similar experiment was made on the Srimuntapor Plot, but in instance nearly double the water actually needed was used in to obtain as much silt as possible; this then gave a duty during al irrigation of 1 cubic foot per second to 38 acres over forty-eight

n the former case, however, the irrigating period was 674 hours, or nty-eight days. Now the works generally are designed to give the e quantity of water but spread over 120 days, hence each cubic foot water from the canal might be made to do $\frac{120}{28} = 4$ times the duty wn in the present experiment, and taken this way, the duty capable eing effected would be 4 × 46 = 184 acres per cubic foot per second; taking an average of the two sets of experiments, of which the latter s of little value, in combination with the former, of 152 acres per c foot per second. But an average of this sort cannot so well be rmined from an isolated plot, as it could be from utilisation of the le of the discharge of a completed distributary. The most useful lt in this case was the absolute amount of water per acre taken n the channels, which was $\frac{2885006}{54} = 53\,406$ cubic feet in the first , and very nearly double that in the second

The Unirrigated Crops of Berar.

	Usual date of sowing.	Shoots after	Buds after	Crop out after	Produce per excluding &c.
The Jarayat Kharif, or early dry crops.		Days.	Days.	Days.	Average.
					lbs.
† Cotton, Gossypium herbaceum		5	120	150	100
† Jowari, Holcus sorghum		7	120	150	300
† Bajri, Holcus spicatus		4	90	105	300
Til, Sesamum oriental		7	90	105	200
† Rice, Oriza sativa		5	60	105	200
Ambari, Hemp ...		3	90	120	80
Baru, Flax		2	60	90	100
† Bhadti		5	60	75	120
Muth		5	90	105	80
Holag		5	90	120	80
* Udidh		7	90	105	240
* Mug, Phaseolus mungo	10 July	4	105	120	300
* Tur	...	5	90	120	180
† Ginger, Zingiber officinale	July	12	700	1000	1100
Red pepper, Capsicum annuum.					
The Jarayat Rabbi, or late dry crops.					
† Wheat, Triticum vulgare	22 Sept.	5	105	135	200
† Tobacco, Nicotiana tabacum	Sept.	8	90	150	200
Kardi	25 Sept.	5	90	135	120
Lakh	9 Oct.	5	105	135	160
Gram, Cicer arietinum					160
Juwas					80
Masur, Ervum lens					80
† Vutanu					160
Gadmol					80

Rough data of increase of yield to the above crops by irrigation.

Jowari, one half more.	Rice, four times more.
Bajri, one quarter more.	Wheat, } one quarter more.
Til, one half more.	Gram, } one quarter more.

* Supplementary crops, sown among others.
† Crops that may be assisted by irrigation.

[redacted]

The Irrigated Crops of Barar.

ayat or Wet Crops grown on d perpetually irrigated or pt damp by rain.	Usual date of sowing.	Shoots after	Buds after	Crop cut after	Produce per acre, excluding straw, &c.	
		Days.	Days.	Days.	Average. lbs.	Max. lbs.
ze, Zea mays	...	5	75	105	100	...
per, Capsicum perennium	1 July	7	105	370	2000	...
gan or Brinjal	"	7	120	370	4000	...
imug	"	5	90	120	800	...
ja, Cannabis sativa ...	"	8	150	150	1600	...
on, Allium cepa	25 Sept.	7	37	120	...	...
lic, Allium sativum ...	"	5	37	120	...	...
hi, Trigonella fenugræcum	"	7	30	120	...	...
rots, Daucus carota ...	"	8	75	75	...	...
d	"	8	135	135	1200	...
um, Papaver somniferum.	1 Nov.	5	75	90	10	20
gmurla	"	5	75	90	...	...
gura	"	5	90	120	240	...
eat, Triticum vulgare ...	"	5	105	120	300	...
ar-cane, Saccharum officinarum	March	12	300	300	1600	7500
g of Goor	"	7	37	75	...	...
nd	"	7	40	80	...	...
li	"	7	75	90	...	...
ai	"	8	90	120	...	...
wala	"	5	90	120	...	...
wala	"	5	37	75	...	...
ntain	23 May	3	360	450	400 trees.	
, Piper betel	...	...	...	...	...	...
it trees	...	...	...	...	...	...

Well Irrigation in Berar.

1. The following crops are watered daily in the hot season, and intervals of from one to seven days throughout the rest of the ye as required; sugar-cane, pan, plantain, bengau, sag, bhaji, and gre vegetable produce; when the sugar-cane is one foot high, the suppl of water is reduced.

2. The following crops are watered once in three days in the ho season, and at intervals of fron seven days throughout the rest of the year as required: m, onions, garlic, perennial pepper, bhoimug, fenugreek, c , chika, chakut, sangchawali, and the common produce of ble gardens.

3. The following crops ar e in three or four days at all seasons, generally: anise, s ric, ginger, ratalu, goradu, pendia, wangi.

4. The following crops s once a week generally: sag of goor, bhend, karli, turai, ala, sangmurla, and rajgura.

5. The remainder are: wa fifteen days; maize, three waterings to the crop; young fruit trees, once a week; older trees, four or five times a year.

The ordinary condition of the irrigation in Berar, is thus:—

The wells have an average depth of 30 feet, and are each worked by one pair of bullocks for nine hours daily, which raise a leather bag (mot) containing 300 lbs. of water. They can thus water half an acre daily well, but for a continuance cannot keep watered more than 3 acres of ordinary irrigated crops. The prime cost of a common unreveted well is £30, the bullocks £15, gear £5, in all £50: the daily expenditure is, feed of bullocks 1s., labour of two men, at 1s. each, in all 3s.; or about £50 a year.

Produce of Crops at the Experimental Farms in Berar, 1870.

Yield of clean cotton in lbs. per acre.

	Umraoti.	Sheagaon.		Umraoti.	Sheagaon.
Banni	184	86	Hinghanghat ...	180	56
Jarri	66	150	Dharwar	14	24

Manured land yielded 430 lbs. of clean cotton per acre.

The following were the yields of other crops:—Jowari, 538 lbs.; wheat, 745; gram, 312; muth, 300; linseed, 278; peas, 408 lbs.

In ploughed land, jowari yielded 660 lbs.

Crops of the Madras Presidency and their Seasons.

al name.		Sown in	Cut in
am...	Sorghum vulgare	September	December.
abu...	Penicillaria spicata	April	June.
ai...	Penisetum italicum	September	January.
nai...	Panicum miliaceum	July	January.
mbai	Triticum vulgare	July	December.
kai...	Zea mais	July	October.
lu...	Oriza sativa...	July	October.
yarai	Cajanus indicus	July	April.
alsai...	Cicer arietinum	July	April.
adu...	Phaseolus aureus	July	February.
a payaru	Phaseolus mungo	September	December.
ani...	Pisum arvense	September	December.
apair	Phaseolus aconitifolius.	December	March.
i	Indigofera tinctoria	November	March.
jel...	Curcuma longa	August	February.
e	Zingiber officinale	September	February.
urchai	Rubia cordifolia...	October	February.
ower	Carthamus tinctorius...	November	March.
kasa	Papaver somniferum	October	March.
rielli	Nicotiana rustica	January	April.
ti	Gossypium herbaceum	May...	January.
a	Cannabis sativa	Six months at any time.	
arai	Corchorus capsularis		
varai	Linum usitatissimum		
panar	Crotalaria pincea	August	March.
hi	Hibiscus cannabinus	August	March.
munak	Ricinus communis	August	November.
agu	Sinapis, three varieties	September	February.
a	Sesamum orientale	January...	April.
milli	Coriandrum	December	March.
ni...	Cucurbita maxima	July	December.
elam	Tricosanthus	July	December.
verai	Trigonella fœnugræcum	July	October.
apallam...	Citrullus	February	April.
ri...	Cucumis sativus	April	July.
m...	Cucumis melo	April	July.
akupi	Anethum sirwa	December	March.

arious sorts of rice are grown in the Madras Presidency, one is a col
and another is left a long time standing; but that above-mentioned is
rop, its period being coincident with the rainy season.

WATER RATES AND WATERINGS.

The Panjab.

On the Bari Doab Canal, from 1862–63 to 1869–70.

		s.	d.
	For all crops, per acre per crop ... 2r. 6a. 8p. or	4	10

Lift irrigation, one-half ate.

0.

		s.	d.
I.	Sugar-cane, per acre	12	0
II.	Rice, per acre per cro ... ... Garden produce, per ear	9	6
III.	Kharif crops. Cot igo, turmeric sesamum, waternuts, ards, fruit trees ... Rabbi crops. Whe ed grain, linseed, sarru, taramira, mustard, opium, tobacco, tukhmbalanga, safflower, chillies, vegetables, per acre per crop	5	0
IV.	Kharif crops. All millets, maize, and crops, not elsewhere mentioned ... ... Rabbi crops. All pulses, all grasses, fallow lands, and crops not elsewhere mentioned, per acre per crop ...	3	0
V.	Single waterings, and Rabbi crops not requiring water after December, per acre per crop	1	6

For lift irrigation, one-half the above rates.

Average supply per acre, 44 000 cubic feet.

On the Western Jamna Canal, from 1862–63 to 1866–67.

	s.	d.
On all first class lands, per acre per crop	2	3½
On all second class lands, per acre per crop	1	4

For lift irrigation, two-thirds the above rates.

Since 1866–67 the rates have been identical with those of the Ganges and Eastern Jamna canals.

On the Delhi and Gurgaon Irrigation Works, from 1862 to 1870, the rates were for grass crops, per acre, 5d.; and for all other crops, per acre, 9½d.

The North-west Provinces.

anges and Eastern Jamna Canals, from 1862–63 to 1865–66.

	s.	d.
Sugar-cane, per acre per year	8	9½
Fruit, nursery and vegetable gardens, all cultivated grasses, rice, waternuts, ajawen, and similar herbs, per acre per crop	5	0
Indigo, cotton, tobacco, wheat and oats (Rabbi), per acre per crop	3	4
. Barley, all pulses and millets, maize, safflower, oil seeds (Kharif), per acre per crop	2	5

From 1865–66 to 1867–68.

Gardens and all lands, taking a perennial supply, were transferred om Class II. to Class I ; and the rates then became for Class I., . 0d.; II., 6s. 0d.; III., 4s. 6d.; IV., 3s. 4d.

Since 1867–68, the fruit, vegetable, and nursery garden produce e been transferred again into Class II., but the rates for the various sses have otherwise remained the same as before. For lift irriga- , the rates have always been two-thirds of those by flow.

The other sources of revenue are, for watering cattle, 12s. per 100, year ; sheep and goats, 4s.; supplying tanks, rent of corn mills, e of grass, timber, fuel, and fruit, fines for trespass, &c.

Dun Canals, from 1862–63 to 1865–66.

For garden produce, sugar-cane, and first-class rice, 2s. 6d. per acre crop ; for tea, 1s. 3d.; for wheat and inferior rice, 1s. 0d.

From 1865–66 to 1867–68.

		per acre. s.	d.
I. Tea, sugar-cane, garden, and perennial watering,	per year	10	0
II. First-class rice, tobacco, opium, and waternut,	per crop	6	0
I. Indigo and cotton	per crop	4	6
V. Inferior rice, wheat, oats, and other crops ...	per crop	2	6

From 1867–68 to 1871 72, tea and sugar-cane remained in Class I., garden and orchard produce being transferred to Class II.; bu rates for the various classes remained unaltered.

Since 1871–72, the rate for tea has been altered to 1s. 6d. for ea watering; leaving sugar-cane alone in Class I.; the rates for oth produce on some of the Dun canals has been lowered.

For lift irrigation, the rates have been always two-thirds of tho by flow.

Rohilkand Canals

			per a
			s. d
I.	Garden and orchard	per crop	4 0
II.	Sugar-cane, tobacco, opium and waternut,	per first watering	1 6
III.	All cereals, pulses, and c ...	per first watering	0 6

In Classes II. and III., hal very subsequent watering

For lift irrigation, the rate f those for flow.

The number of watering n the Naginah Canal is:—

For fruit gardens	per year	8 waterings.
Hemp	per crop	5 "
Rice, sugar-cane, ind cultivated grasses	per crop	4 "
Cotton, cereals, and p	per crop	3 "

NAVIGATION TOLLS IN NORTHERN INDIA.

The Western Jamna Canal transit dues are tabulated according to a most complicated code, the rates for various sorts of timber varying from 1s. 3d. to £4 per score for the whole course of the canal, with a reduction for intermediate distances; the rates by weight being about 6d. per ton for the whole course of the canal.

The Bari Doab Canal transit dues are:—

For rafts of all sorts of timber ...	1⅛d. per £10 value at starting.
For rafts of bamboos	⅜d. per thousand.
For rafts of firewood, hemp, flax, and grass	⅛d. per 4 tons, or 100 mans.
For rafts of reeds, sirkanda ...	⅛d. per thousand bundles.

The Ganges Canal transit dues, since 1872, have been:—

		s.	d.
For boats, per month		9	0
Rafts of logs, per mile	per 100 cubic feet.		1⅛
Rafts of sleepers, &c., per mile ...	" "		⅛
Rafts of bamboos, per mile	" "		¼
Rafts of firewood, per mile	per 1000 "		⅛

The Eastern Jamna Canal is very little used for navigation.

WATER RATES AND WATERINGS IN SOUTHERN INDIA.

n *the Bombay Presidency* there is generally a combined land and gation assessment. The canals are divided into three sorts, and sified according to depth of soil, in cubits of 18 inches, and with pect to their special advantages and disadvantages. No advantage considered to arise from more than two cubits in depth of soil, as it not imbibe and retain more effective moisture; the disadvantages n into consideration are the presence in the soil of kankar, coarse d, loose or stiff soil, excess of moisture, and liability to be flooded. a moist climate the better and worse descriptions of land are conred more on a par, the latter benefiting more from moisture than former.

The general assessment, per acre, is as follows:—

	s.	d.		s.	d.
For unirrigated or dry crops	3	6			
For ordinary irrigated or garden crops ...	8	0			
For special irrigated crops in some places ...	14	0	to	30	0

The rates allowed on the Mukti project are.—

For sugar-cane, 56s.; for rice, 20s.; for wheat, 10s. per acre.

And those allowed on the Lakh project and Bhatodi tank are:—

For perennial, or 12 months, irrigation,	per acre	18s.
For wet and cold season, or 8 months' irrigation	„	10s.
For mansun, or 4 months' irrigation ...	„	6s.

The amount of watering considered necessary per square yard of ivation is:—

For rice crop ...	4 months ...	½ cubic yard.
For sugar-cane ...	11 months ...	1 „

A good well will keep irrigated from four to six acres of inferior den crop.

n *the Madras Presidency* there is generally a combined land and gation assessment. The consolidated revenue, including the water e, is two-fifths of the value of the produce, but is sometimes less, ording to the market price of rice.

The general assessment, per acre, is as follows :—

	s.	d.		s.
For unirrigated or dry crops	4	0		
For rice	9	6	to	16
Sugar, at the same ratio, would be sometimes as much as				120
But the general range of assessment is from... ...	4	0	to	50
The water rate allowed by Government on the Tumbaddra Canal of the Irrigation Company is ...	10	0	to	12
In Maisur, the general rate per acre is	12	0	to	13

The general allowance of water [illegible] ce crops in the Madras Presidency is 1 cubic foot per second [illegible] dy to 40 acres; to sugar cane, gram, plantain, and garden c[illegible] to 120 acres; ordinary field crops are rarely grown in place[illegible] rrigation is available.

When comparing the water [illegible] vogue in different parts of India, the average wages of [illegible] bourer, or coolie, should be borne in mind. The followi[illegible] oximate data :—

In Northern India	3d. to 4½d.
In Barar	6d. to 9d.
In the Bombay Presidency	6d. to 9d.
In the Madras Presidency	2¼d. to 3¾d.
In Maisur	3d. to 6d.

DESCRIPTION AND ANALYSIS OF WATER.

ion of Silt per 100 000 *parts of water brought down by various rivers.*
(*Reduced from Heywood's table.*)

River.	Mean Proportion		Maximum.	Minimum.	Authority.
	By bulk.	By weight.	By weight.	By weight.	
ssippi at Carrolton...	29	55	...	...	Miss. D. Survey.
at Columbus ...	40	76	...	...	„ „ „
at the mouths...	42	80	...	...	Mr. Meade.
„ ...	31	58	...	...	Mr. Sidell.
, at New Orleans	33	87	...	...	Prof. Ridell.
... ...	...	156	...	...	Mr. Horner.
n 1874	...	...	[illegible]	7	Mr. Fowler.
w River, China ...	...	333	...	...	Sir G. Staunton.
ges	98	190	...	...	Mr. Everest.
hli, at Calcutta .	...	68	136	25	Dr. Macnamara.
s	...	...	476	...	Col. Tremenheere
raddi	...	33	58	17	Mr. Login.
... ...	333	...	...	...	Mr. Tadini.
ne, at Lyons	...	6	...	...	Mr. Surell.
at Arles	...	50	435	14	M. Subour.
at Bonn	...	6	8	5	Mr. Horner.
nne	13	19	75	...	Mr Baumgarten.
se	...	10	21	0·7	—
e, at Rouen	...	2	4	...	M. Marchal.
e	...	4	10	0·8	—
ube	8	...	...	...	M. Marchal.
, at *Ely*	...	1	4	0·1	Mr. Latham.

Analysis of the Water and Silt of the Nile in 1874 by Dr. Leth

Constituents per 100 000 parts.		June 8.	July 10.	August 12	Sept. 20.
Actual or saline ammonia ...		0·0057	0·0129	0·0048	0·0100
Ammonia from organic matter		0·0114	0·0100	0·0071	0·0171
Dissolved Matters.	Lime	[illegible]	3·993	4.422	4·260
	Magnesia	[illegible]	5·113	1·030	0·617
	Soda	[illegible]	0·744	0 587	0·301
	Potassa	[illegible]	1·062	1·501	4·120
	Chlorine	[illegible]	0 851	0·628	0·209
	Sulphuric acid ...	[illegible]	2 838	1·837	1·996
	Phosphoric acid ...	[illegible]	...	...	...
	Nitric acid	trace	...	...	...
	Silica alumina and oxide of iron	0·701	0 713	1 129	1·257
	Organic matter	1 500	1·057	1·186	1·029
	Carbonic acid and loss ...	4·182	3·616	4·281	4·754
Total solid matter on evaporation		20·300	16·386	16·601	19·443
Suspended matter	Organic	0·829	9·114	18 414	5·914
	Mineral	6 086	8·729	130·743	48·343
Total suspended matter ...		6 915	17·843	149 157	54·257

The average percentage of the sedimentary deposit from all the above was:—

Organic matter.	14·61	Potassa	
Phosphoric acid	1 78	Soda	
Sulphuric acid	trace.	Alumina	
Chlorine	trace.	Peroxide of iron	
Lime	2·06	Silica	
Magnesia	1·12	Carbonic acid and loss ...	

The Nile water owes its fertilizing power not only to the quantity of a nitrogeneous organic matter, the soluble silicates of potassa and soda, and of phosphoric and of nitric acid in the water, but also to the sedimentary which are charged with phosphates and alkaline silicates.

Analysis of the Water of the Ganges and its Affluents.

Date ... ... Place	1. Apr. 1867, near Allahabad.	2. 26 May, above Danapur.	3. May, 1867, below Khanpur.	4. 4 Oct. 1867, below the Son, Danapur.	5. 21 Oct. 1867, at Allahabad.	6. 14 Nov. 1867, at Khanpur.	7. 1 May, 1869, at Fattahgarh	8. 11 Sept. 1869, ½ mile above Chunar.	9. Bhagaratti. 26 Oct. 1867, opposite Barhampur	10. Son. 6 Oct. 1868, at Danapur.
Total hardness...	5·8	6·	4·3	5·8	8·20	4·5	3·7	7·0	5·35	3 5
Permanent hardness	2·5	3·25	3·5	3·9	3·2	3·2	1 8	3 1	2·73	2·8
Grains of oxygen required per million grains	·62	·016?	7·3	·035?	·48	·4	·7	·45	1·07	·61
Ammonia	present	...	present	...	...	0	none	traces	...	traces
Phosphoric acid	abund.	present	abund.	...	...	0	none	none	traces	traces
Nitrous acid	...	...	0	...	...	0	none	none	...	traces
Grains of nitric acid in 70 000 ...	...	traces	0	...	...	0	none	none	...	under 1gr
Total solids in 70 000 grains of filtered water...	11·9	10 9	11·06	14·3	8·4	9·2	9·1	8·75	13·05	10·22
Volatile matter	3·5	1·05	2·52	2·8	·7	·51	1 75	1 4	1·26	3·01
Mineral matter	8·4	9·85	8·54	12 1	7·7	8·69	7·35	7·35	11·70	7·21
Earthy salts, silica, oxide of iron, insoluble	4·06	6·8	5·25	7·0	5·25	7·4	4·37	6·65	8·9	5·25
Lime, as carbonate	2·9	6 7	2·52	5·1	3·15	?	3·29	4·9	8·7	3·78
Silica	...	traces	traces	traces	...	...	·7	traces	3·15	1·68
Soluble salts	4·34	3 05	3 29	5·1	2·45	1·29	2·97	·7	2·8	1 96
Chloride of sodium	1·05	1·05	·8	1·20	1·05	·42	1·05	·74	·63	·42
Sulphate of soda	1·5	2·68	1·54	2·34	?	?	1·92	traces	·4	·45
Carbonate of soda	2·0	1·07	·9	1·5	·47	·76	...	70	·6	·44

1 and 3, by Dr. Milne; 2 and 4, Dr. Jameson; 5 and 6, Dr. Compigne; 7 and 8, Dr. Whitwell; 9, Dr. Thomson; 10, Dr. May.

The Ganges is believed to supply the best river water in India.

Analysis of the Water of the Jamna.

	1.	2.	3.	4.	5.	6.	7
Date. ..	28 Sept. 1866,	14 Dec. 1866,	25 April, 1867,	April, 1867,	17 May, 1867,	23 Oct. 1867,	28 June, 1868,
Place	above Delhi.	2 miles above Agra.	above Agra.	opposite Allahabad.	above Delhi.	at Allahabad.	1½ mile below Mattra.
Total hardness	4·45	6 7	8·0	8·8	4·7	8·78	4·1
Permanent hardness	2 86	2 95	1·0	4·6	3·95	2 36	2·0
Grains of oxygen required per million grains ...	·05	·35	·48	·715	·055	·6	·26
Ammonia	...	...	...	traces	0	...	...
Phosphoric acid	...	...	present	present	0	...	...
Nitrous acid	...	...	...	...	0	...	...
Grains of nitric acid in 70 000	...	...	...	...	0	...	...
Total solids in 70 000 grains of filtered water ...	11·4	14·8	16·8	21·0	10 04	11·2	13·3
Volatile matter	·72	1·2	2·8	3·5	·34	·35	·8
Mineral matter	10 92	13 16	13·3	17·5	9 7	10·85	12·5
Earthy salts, silica, oxide of iron, insoluble ...	8·62	7 64	5 53	9·1	7·16	7·01	7·0
Lime, as carbonate	3·73	6·4	unknown	4·27	4·9	0·5	4 2
Silica	·58	traces	...	...	·63	...	traces
Soluble salts	2·29	5·96	7·77	8·9	3·54	2·04	4 9
Chloride of sodium	·84	1·44	2·1	4.4	72	1 6	1 37
Sulphate of soda	1·15	1 6	unknown	3·6	2·6	?	2·

Analysis of the Water of the Rivers of the Panjab.

Date ... ... / Place	1. Indus. 23 June, 1868, at Attock.	2. Indus. 24 Dec. 1868, at Attock.	3. Indus. 28 Apr. 1869, at Dera, Ismail Khan.	4. Cabul. May, 1868, at Nau-shera.	5. Cabul. 24 Dec. 1868, 1 mile above Nau-shera.	6. Cabul. Jan. 1870, near Fort Michni.	7 Ravi. 16 Dec. 1868, at Mian Mir?	8. Jhelam. 10 May, 1869, 1½ mile below Rawal-pindi.	9. Satlaj. 28 Mar. 1870, at Bhawal-pur.	10. Gaggar. 21 Dec. 1867, at Muba-rikpur.	11. Gaggar. 28 Nov. 1868, 8 miles from Amballa	12. Harru. 13 Oct. 1867, above Camp-bellpur.	13. Harru. 24 Nov. 1868, 1½ mile above Saidan Baoli.	14. Leh. 25 Sept. 1867, above Rawal-pindi.	15. Swat. January 1870, near Abazai.
Total hardness	3·3	6·5	4·8	4·2	8·8	10·72	5.95	2·22	7·16	6·3	6·9	8·5	9·63	8·7	6·6
Permanent hardness ...	4·3	5·	2·5	2·8	5·2	4·75	2·92	1·27	3·55	2·5	3·7	6·7	6·7	4·7	2·2
Grs. of oxygen required per million grains	·51	·16	·69	·51	·1	·80	·29	1·38	·25	·24	·29	·5	.19	·39	·24
Ammonia	...	...	none	present	...	none	none	none	none	...	...	...	...	traces	trace
Phosphoric acid	traces	...	none	traces	traces	trace	traces	none	traces	traces	traces	...	...	present	trace
Nitrous acid	...	...	none	...	...	trace	none	none	none	...	...	...	...	present	none
Grs. of nitric acid in 70 000	...	traces	none	...	...	none	none	...	none	...	...	traces	·5	traces	none
Total solids in 70 000 grs. of filtered water	5·14	10·15	10·73	9·7	15·75	18·1	11·70	7·7	11·85	15·2	15·48	17·4	17·8	19·5	10·45
Volatile matter	·35	·42	·81	·45	·14	1·60	·64	·7	·40	·34	·7	1·2	·3	1·5	1·4
Mineral matter	4·79	9·73	9·93	9·25	15·61	16·5	10·06	7·0	11·45	14·88	14·78	16·2	17·5	18·	9·05
Earthy salts, silica, oxide of iron, insoluble	3·7	5·30	4·64	5·07	...	11·2	8·70		6·15	10·9	9·7	...	...	14·	7·25
Lime, as carbonate	3·0	...	3·04	3·46	...	3·75	4·70	analysis unfinished.	4·40	8·4	6·3	...	11·1	8·6	4·5
Silica	·3	...	unk.	...	...	present	·80		·75	...	·88	...	...	2·31	traces
Soluble salts	1·19	4·34	5·29	4·18	...	53 ?	2·27		5·30	3·8	5·07	...	...	4·	1·8
Chloride of sodium	·21	·42	1·31	·6	·8	1·45	·32		1·20	·42	·75	·42	·49	·63	·45
Sulphate of soda	1·3	2·6	1·35	2·64	3·6	4·1	1·47		3·30	2·6	2·23	...	...	·8	1·4
Carbonate of soda	·72	1·4	·24	1·56	...	...	...		·95	1·3	2·5	·6	...	·9	·27

1, 2, 4, 5, 10, 11, 12, 13, and 14, by Dr. Center; 7, 10, and 11, Dr. Sheppard; 3, Dr. Thomson; 6, 8, and 15, Dr. Harvey; 9, Dr. Hutcheson. 1, water rising rapidly, nearly at its highest; 2, river at its lowest; 5, river at its lowest.

Analysis of the Water of the Rivers of Oudh, Rohilkand, and the Panjab.

	1	2.	3.	4.	5	6.	7.	8.	9	10.	11	12.	13.	14.	15	16.
	Gumti	Gumti.	Gogra	Gogra.	Sai.	Sai.	Surain.	Surain	Kurnaul.	Garrah.	Ram ganga	Ram-ganga.	Gangutr	Tovey	Kurram.	Bain [illegible]
Date	22 April, 1867,	26 Dec 1868,	16 June, 1867,	11 June, 1868,	27 April, 1868,	20 April, 1868,	8 April, 1869,	16 Jan. 1869,	27 May, 1869,	5 June, 1869,	6 July, 1869,	27 July, 1869,	29 July, 1869,	3 Oct 1870,	14 Nov 1870,	
Place	above Lakhnau	at Lakhnau	1½ mile above Fyzabad	1½ mile above Fyzabad	2 miles below Rai Bareli	opposite Rai Bareli.	at Sitapur	at Sitapur	1 mile above Shahjahanpur.	3 miles above Shahjahanpur.	at Bareli	¼ mile above Moradabad	4 miles above Moradabad	2 miles above Kohat	9 mi from Bannu.	
Total hardness	4·8	4 7	4·5	4 2	8	8 02	7 2	6·7	8 53	10·28	3·9	3·25	4·4	15 9	6 81	3 1
Permanent hardness... ...	2 09	2·	2·6	2 7	3·2	3·2	4 2	3·8	2·54	5·2	2 86	3·15	2·9	8·27	7·11	2·04
Grains of oxygen required per million grains... ...	·11	·11	·08	...	1	·2	...	[illegible]	2·0	·65	·25	·4	·3	·19	·28	59
Ammonia	0	0	...	...	...	...	traces	[illegible]	[illegible]	[illegible]	[illegible]	none	none	none	none	none
Phosphoric acid	0	0	...	...	...	...	...	[illegible]	[illegible]	[illegible]	[illegible]	none	none	none	none	none
Nitrous acid	0	0	...	...	..	...	...	[illegible]	[illegible]	[illegible]	[illegible]	none	none	none	none	none
Grains of nitric acid in 70 000	0	trace	...	...	traces	freely present	present	[illegible]	[illegible]	[illegible]	[illegible]	none	none	none	...	...
Total solids in 70 000 grains of filtered water	15·4	14·	11 2	10 85	16 8	15 4	17·0	16·1	18·76	17·67	14·87	18·37	14·0	29·4	16·8	5·6
Volatile matter	1·4	1·05	·84	·7	1·4	1 75	2·0	1·4	1·90	1·93	2·21	1·75	·87	1·26	1 4	·7
Mineral matter	·14	12 95	10 36	10·15	15·4	13 65	15·	14·7	16 8	15·75	12 67	16·63	13·13	28·14	15·4	4 9
Earthy salts, silica, oxide of iron, insoluble	9·52	8·57	8·2	8 75	4·9	4·9	10·8	11·2	12·42	8·75	9 27	14·7	11·03	14·14	7·	2·8
Lime, as carbonate	7·0	8·05	5.	8·05	4·2	4·2	9 9	10·85	6·68	5·69	5·07	7·0	49·55	9 8	5·40	1·61
Silica	trace	0	...	traces	·21	·18	·3	trace	3·5	2·27	3·5	5·25	4·9	·98	trace	...
Soluble salts	4·48	4·37	2·16	1·4	10 5	8·75	4.2	8·5	4·37	7·0	3·39	1 03	2·1	14	8 4	2·1
Chloride of sodium	1·4	·63	1 7	·03	6 3	5·2	·8	·75	1·20	1·06	·74	·63	·74	2·52	1 08	1·05
Sulphate of soda	trace	·58	traces	traces	1·9	1·6	2 2	1·48	2 80	11·23	7 60	3·58	3 58	4 30	2 10	traces
Carbonate of soda	2 28	2 28	·6	·48	1·3	1·0	1 4	1·24	3 52	3·43	nil.	·57	1·38	5 6	3·78	·76

1, 2, 3, 4, 5, 6, 7, 8, by Dr Orton, 9, 10, 11, 12, 13, 14 15, and 16, Dr Whitwell
4, very good; 7, indifferent water, after a heavy rainfall.

Date ... ... Place	17 June, 1867, Marble rocks.	27 April, 1868, 5 miles from Jabalpur.	28 June, 1870, at Jabalpur.	26 Sept. 1867, 1 mile below town	16 July, 1868? above bend.	13 August, 1868, ? above bend.	7 Feb. 1870, 5 miles above Morar-Bazar.	3 April, 1868, above Nagod.	14 Nov. 1867, 5 miles from Jhansi.
Total hardness	6·9	8·5	3·3	5·1	5·9	5·4	7·7	13·3	3·4
Permanent hardness	3·6	2 5	2·3	·9	·6	1·2	5·0	2·4	1·7
Grains of oxygen required per million grains	·67	·4	·315	·6	·355	·21	·46	·59	·215
Ammonia	0	...	none	...	present	...	none	...	...
Phosphoric acid	present	present	none	present	present	present	none	...	trace
Nitrous acid	0	...	none	trace	present	...	none	...	...
Grains of nitric acid in 70 000	...	...	none	·5	trace	...	none	...	...
Total solids in 70 000 grains of filtered water	12·16	11·5	5·35	10·3	15·54	9·8	15·75	18·9	9·8
Volatile matter	2·8	2·6	·95	1·2	1·3	1·3	·91	1·9	1·
Mineral matter	9·38	8·9	4·40	9·1	14·24	8·5	14·84	17·	8·8
Earthy salts, silica, oxide of iron, insoluble...	...	...	3·60	4·9	...	5·	8 4	10·	4·7
Lime, as carbonate	9·8	5·6	1·10	3 2	6·0	3·3	5·74	8·8	2·8
Silica	...	1·4	1·35	traces	2·13	...	2·1	·42	...
Soluble salts	2 1	...	·80	4·2	...	3·5	6·44	7·	4·1
Chloride of sodium	·26	1·05	·30	1·2	3·01	1·5	1·47	1·7	1·15
Sulphate of soda	traces	...	·365	...	·47	1·6	5·11	1·9	...
Carbonate of soda	·25	·51	·27	1·3	2·03	...	traces	3·	1·14

1, by Mr. Griffith; 2 and 8, Dr. Thomson; 3, Dr. Hutcheson; 4, 5, 6, and 9, Dr. May; 7, Dr. Whitwell.

Analysis of the Water of various Canals.

	1.	2	3	4.	Canals of the Deyrah Dun.				9.	10.	11
	Canal from the Ganges. Apr 1867, below Khanpur.	Canal from the Ganges. 11 Nov. 1867, above Khanpur.	Canal from the Ganges 23 Aug 1869, 3 miles above Allighar.	Ganges Canal. 1 April, 1870, below Rurkhi Aqueduct.	5. 27 Dec. 1869, Main 2 miles above Dera.	6. 7 Jan. 1870, Branch.	7. 11 Jan. 1870, Reservoir Branch.	8. 2 Feb. 1870, Branch ½ mile above Gurka Lines.	Canal from the Ravi. 18 Dec. 1868, at Mian Mir.	Canal from the Barra. 19 May, 1867, above Peshawar.	Canal from the Kurram 17 Nov 1870, near Fort of Bannu.
Total hardness	4·35	4·7	3·2	2·14	18 1	18·63	18·42	18·32	5·2	7·63	7 0
Permanent hardness	2·86	2·8	3 2	1·31	11·94	11·00	11·77	11·27	3·7	3 45	6·9
Grains of oxygen required per million grains	·3	·65	·45	[illegible]	[illegible]	·345	·296	·255	·25	·825	·535
Ammonia	present	0	none	[illegible]	[illegible]	[illegible]	[illegible]	none	none	present	present
Phosphoric acid	large	0	none	[illegible]	[illegible]	[illegible]	[illegible]	none	traces	traces	none
Nitrous acid	traces	0	none	[illegible]	[illegible]	[illegible]	[illegible]	none	none	present	present
Grains of nitric acid in 70,000	0	0	none	[illegible]	[illegible]	[illegible]	[illegible]	traces	none	...	...
Total solids in 70,000 grains of filtered water	6·6	6·26	8·03	[illegible]	[illegible]	...	[illegible]	59·92	9·40	12·841	22·4
Volatile matters	·72	·7	·87	·7	·4	...	·6	·42	·48	2·677	1·4
Mineral matters	7 35	7 56	8 05	4·9	52·3	...	52·5	59·5	8·92	10 164	·21
Earthy salts, silica, oxide of iron, insoluble	5·67	5·07	5·78	...	22·4	...	...	...	5·97	6·16	10·6
Lime, as carbonate	3·71	2·6	3·71	2·1	18·7	17·59	18 4	22 66	6·24	unknown	0·58
Silica	traces	...	2·45	traces	3·3	present	9	traces	·4	·6	1 36
Soluble salts	1·6	2 5	2·27	...	29·0	...	...	...	2·05	4·0	10 5
Chloride of sodium	1·2	1·5	·53	1·12	·3	1·30	2·8	2 5	·66	·55	1 68
Sulphate of soda	1·28	?	2·98	none	21·67	28 4	24 95	28 75	2 52	1·8	5·04
Carbonate of soda	·33	?	·66	none	1·36	...	...	8 20	...	·126	8·09

1, by Dr. Milne, 2, Dr. Compigne; 3 and 11, Dr. Whitwell, 4, 5, 6, 7, 8, Dr. May; 9, Dr. [illegible]; 10, Mr. [illegible] 4, [illegible] *of Analysis of the average Well Waters of Stations in Northern India, according to various Analysts.*

ation.	Date of Examination.	Grains per gallon, or parts in 70 000.			Oxygen required per million parts.	Character and Remarks.
		Total Solids	Volatile matter.	Chlorides.		
awar ...	May 1868	27·4	2·7	2·0	0·50	Indifferent.
hera ..	May 1868	18·2	0·98	0·8	0·27	Very wholesome.
k	May 1868	123·3	3·8	26·0	...	Very bad.
lpindi ...	Sept. 1867	28·9	3·5	0·6	0·51	Pure and good.
Mir ...	Dec. 1868	59·3	1·4	3·3	0·63	Very bad.
isar ...	Dec. 1869	56·2	6·1	15·6	...	Good.
mail Khan	Apr. 1868	87·2	1·5	5·8	0·47	Fair.
hazi Khan	Mar. 1869	42·7	1·8	8·7	0·62	Fair.
... ...	...	75·0	7·8	unk.	...	Very bad.
a... ...	Dec. 1867	39·6	2·0	10·8	0·51	Fair.
... ...	Jan. 1868	45·4	4·1	11·2	0·47	Bad.
bad ...	Jan. 1867	18·6	1·3	1·8	0·17	Good.
hgarh ...	Apr. 1869	34·3	2·2	4·6	0·54	Doubtful.
arh ...	Aug. 1869	35·1	2·6	5·7	0·44	Very foul.
habad ...	Mar. 1860	33·1	1·1	3·9	...	Fair, but hard.
eras ...	Dec. 1868	25·9	1·3	2·8	...	Good.
ar ...	Sept. 1869	34·8	1·4	4·3	...	Hard and bad.
pur ...	Sept. 1868	59·2	5·5	10·3	0·31	Very bad.
hampur ...	Nov. 1867	31·1	2·8	8·7	...	Bad.
lpur ...	May 1868	21·0	1·9	4·1	0·76	Wholesome.
i	Nov. 1867	25·1	4·9	2·4	0·53	Wholesome.
ar	Aug. 1868	29·6	2·1	5·7	0·51	Bad

No. of wells ...	5	8	9	14	10	10
Appearance ...	Clear	Clear	Clear	Clear to turbid	Various	Clear
Odour ...	None	None	None	None	None	Various
Taste ...	None	Slightly brackish	Agreeable	Various	Various	Do.
Reaction to test paper ...	Neutral	Alkaline	Faintly alkaline	Acid	Do.	Acid
Hardness, temporary ...	2 to 5½	2 to 13	3 to 8½	½ to 7	½ to 6	2 to 35
Do. permanent ...	14 to 16	7 to 25	9 to 18	5 to 20	7½ to 16	6 to 17
Solid ingredients in one Imperial gallon.						
Organic, in grains ...	3 to 5	3 to 9	4 to 9	2 to 6	2 to 8	4 to 8
Inorganic, Do. ...	60 to 79	27 to 242	24 to 48	10 to 61	10 to 48	19 to 64
Chlorides, Do. ...	40 to 60	7 to 180	10 to 24	4 to 38	3 to 29	4 to 26
Bases in solution by Carbonic Acid.						
Iron ...	Average	Average	Average	Above	Average	Traces
Lime ...	Do.	Do.	Do.	Large	Do.	Average
Magnesia ...	Do.	Do.	Do.	Average	Traces	Do.
In solution after boiling.						
Lime ...	Large	Large	Fair	Average	Fair	Various
Magnesia ...	Average	Do.	Large	Above	Average	Do.
Chlorides ...	Very large	Do.	Do.	Large	Small	Not determined
Sulphates ...	Large	Do.	Rather large	Do.	Fair	Large
Nitrates ...	None	None	None	None	None to abundant	None

N.B.—The well water of Madras is believed to be as bad as possible.

Peshawar.—The drinking water is obtained by open canal from t river Barra, which also fills reservoirs; the water is excellent, b sometimes muddy; the reservoirs are frequently drained, but conta frogs, also Typha angustifolia, Potamogeitons and Confervæ.

The Peshawar Marsh being specially renowned for its malario effects, an account of the flora that thrive there will therefore be o interest. On the higher ground, which is covered with saline effl rescence, grow several species of Salsolaceæ, Franknia pulverulent Tamarix, Salix Babilonica. The ordinary plants that grow in an around the marsh are:—Epilobium, occasional; Lycopus, abundant i parts; Lippia nodiflora and Herpetis monneira, about ditches; Utrica laria, rare; Eclipta erecta, not uncommon; Ranunculus aquatilis an Ranunculus sceleratus, common; Limnanthemum cristatum, a specie of Lium; Typha angustifolia, abundant; Nelumbium, cultivated Butomus, rare; Sagittaria sagittafolia, Alisma equisetum, two specie of Juncus, rare. Of Sedges, the following are common:—Cypicu exaltatus, Cypicus mucronatus, Malacochæte pectinata, Scupus mari timus, Carix Wallichiana, Eleocharis palustris. The common grass about and near the water are:—Agrostis alba, Polypagon monspelien Andropogon Bradlii, Cynodon dactylon, an Arundo, a Saccharum. The following are the floating and submerged plants:—A Ceratophyllum (demersum?), Potamogeiton crispus, P. pusillus, Potamogeiton plantageneus, rare; Hydrilla verticillata, Marsilia quadrifolia, Chara, most abundant; Nitella, occasional; Confervæ, profuse. Two species of Riccia, a Semno, and an Argola, are abundant in some places.

The Well Water of the Stations of the Bombay Presidency.

Bombay.—Well water brackish, containing a large quantity of lime, also sea salt. Vehar reservoir water is considered very pure.

Sattara.—Wells and tanks in trap rock; the guinea worm is found in them.

Malligaum.—The wells require clearing from sediment once a year, and would otherwise become unwholesome.

Belgaum.—Well water clear, good, soft and wholesome, contains chlorides, sulphates of lime and magnesia, and a salt of iron. Free from taste and smell.

Ahmadabad.—The well water, after long use, is apt to induce disease of the spleen, which the river water does not; the former has a higher specific gravity than the latter.

roda.—Well water clear, soft, and of good quality; it contains lphates, phosphates or nitrates, nor any salts of lime; it is ine ;—it contains principally chloride of sodium, also carbonate oda, and a faint trace of lime, but no iron.

ssirabad.—Most of the wells are so salt that they are unfit for use. water from the same well varies considerably in saltness, being times palatable, clear and hard; that from a wholesome well was d to contain, after evaporation to dryness, organic matter in the e proportion of 1 in 200, as well as chloride of sodium and sulates of alumina and potass, besides other chlorides and sulphates.

eesa.—Well water clear, agreeable, devoid of smell, almost free organic matter, with an inconsiderable amount of saline or eral ingredients.

holapor.—Wells supplied by percolation from the tanks; water good, soft, pure, uninjurious, and colourless, when filtered has a ific gravity of 1000·4 and contains 30 grains of solid matter to a on : under microscopic examination was found to contain no organic tter beyond a little shiny film. The tanks contain Flosaquæ, as well ordinary grasses and rushes, and among the infusoria the encapd amalæ oscillatoria, and ædogonium; in dry weather, when the decomposes, the malaria is most noxious.

rat.—There is not a single well fit for drinking from within the tion. All are impregnated with sulphuretted hydrogen.

yderabad in Sind.—The wells are supplied by inundation from the dus. The water is said to be soft, good and wholesome, a few ls only brackish: yet the wells swarm with animal life. Like t wells in Sind, they may be exhausted by an ordinary Persian eel in twelve hours.

Dharwar.—The well water has the reputation of being very good and holesome, but also to give rise to guinea-worm among the natives.

Dhulia.—Well water good, soft, devoid of smell, of an agreeable te, but of a rather blue colour.

Serur.—Well water hard, but good and wholesome; it contains a tle lime.

Ratnagherri.—Well water very good, as soft as rainwater, and free m taste or smell.

END OF HYDRAULIC STATISTICS.

[redacted]

INDIAN

METEOROLOGICAL STATISTICS

FOR THE USE OF ENGINEERS.

……	BURDWAN.						PRESIDENCY.				RAJ SHAHYE.						COOCH BAHAR.			DACCA.				
Place……	Bardwan.	Raniganj.	Bankura.	Suri.	Midnapur.	Contai.	Saugor Island.	Calcutta.	Kishnaghur.	Jessor.	Barhampur.	Dinajpur.	Malda.	Bauliah.	Rangpur.	Bogra.	Darjiling.	Rangbi.	Buxa.	Dacca.	Faridpur.	Mymensingh.	Sylhet.	Silchar.
Height above Mean Sea Level.	102	326	…	157	108	43	6	18	…	20	64	…	160	110	…	…	6912	..	1800	35	…	…	…	89
No. of Years.	13–16	6	15–16	10	7–9	5 6	5–6	32	9–13	11–15	15–17	9–11	15–17	11–13	9–11	10–12	10–13	6	4	12–13	10	8–9	13–17	12–14
January to May.	9·0	5·2	7·7	5·4	10·2	8·8	9·1	10·1	13·8	14·4	8·8	12·5	8·9	10·3	14·1	16·4	14·5	21·5	35·2	19·5	21·5	24·0	44·2	36·9
June to September.	43·4	45·3	40·0	41·3	44·9	50·4	59·9	49·7	37·6	44·8	39·8	65·6	39·4	45·2	68·6	64·7	101·9	144·6	204·0	48·2	48·6	73·1	99·6	73.1
October to December.	6·7	4·2	4·9	4·8	7·2	15·2	13·3	6·2	5·5	7·2	6·4	6·5	5·5	6·2	5·5	7·3	8·5	9·2	17·1	6·7	5·7	5·8	10·5	9·6
Year………	59·	55·	53·	52·	62·	74·	82·	66·	57·	66·	55·	85·	54·	62·	88·	88·	125·	175·	256·	74·	76·	103·	154·	120·

I.—India.—Average Rainfall of 70 Stations, according to the [illegible], [illegible] 1878 [illegible]

District	Assam.					North-west Provinces								Central Provinces.							Ceylon.			
Place......	Naugong.	Sibsagar.	Nazirah.	Shillong.	Cherrapunji.	Banaras.	Gorakpur.	Jhansi.	Lukhnau.	Agra.	Ajmir.	Bareli.	Rurkhi.	Jabalpur.	Nagpur.	Sagar.	Hoshungabad.	Chanda.	Raipur.	Seoni.	Jaffna.	Trincomali.	Colombo.	Galle.
Height above Mean Sea Level.	...	...	...	4825	4460	263	255	836	364	551	...	570	880	1353	1025	1860	1030	690	977	2030	9	175	42	40
No. of Years.	11–13	13–15	11–12	5–6	6–13	9–10	6	5–8	6	9–10	7–8	5–6	9	12	24	16	12	7	7	11	3	3–4	4	4
January to May.	24·4	28·6	24·0	14·3	104·	2·3	3·5	3·3	2·9	2·1	2·4	4·4	6·8	1·9	3·2	2·0	1·3	3·1	2·9	2·9	12·9	19·5	32·6	25·9
June to September.	57·6	58·5	55·3	56·3	400·	34·8	44·0	29·1	41·7	22·7	19·3	36·8	30·7	50·1	38·4	46·5	43·9	43·1	44·8	[illegible]1·4	5·0	9·7	14·5	19·4
October to December.	5·6	7·0	6·8	7·3	19·8	2·1	4·9	2·0	2·7	0·8	0·7	1·8	1·8	1·7	3·7	2·1	1·6	2·3	2·9	2·6	32·1	30·3	33·1	28·5
Year	87·	94·	86·	78·	524·	37·	52·	34·	47·	26·	22·	43·	39·	54·	45·	51·	47·	48·	51·	47·	50·	59·	80·	74·

II.—Bengal.—Mean Monthly Rainfall for 16 Places before 1861.

Place	Puri.	Thayat Myo.	Midnapur.	Calcutta.	Fort William.	Hoshangabad.	Hughli.	Hazaribagh.	Barhampur.	Chunar.	Bhagalpur.	Uttramullay.	Cherrapunji.	Battikot.	Enteebwar.	
Latitude	19° 48′	20° 18′	22° 25′	22° 34′	22° 34′	22° 44′	22° 58′	24° 0′	24° 5′	23° 5′	25° 14′	24° 55′	25° 16′	26° 40′	26° 46′	27° 3′
Longitude	85° 49′	92° 46′	87° 19′	88° 25′	88° 25′	77° 44′	88° 26′	85° 24′	88° 14′	[illegible] 0′	87° 0′	[illegible]	91° 44′	91° 59′	79° 5′	88° 18′
Height	15	240	...	81	18	1030	..	1900					4500	...	...	7000
No. of Years	1	1	1	15	5	1	1	3					2	2	1	3
Actual Years	1851	1859	1851	1836–50	1855–59	1849	1851	1858–60					1859–60	1847–48	1848	1857–59
January	·0	·0	·3	·4	·3	1·5	1·0	·0	·1	·5	·8	4·3	·0	1·5	·0	·0
February	·4	·0	·2	·6	·4	·0	·6	1.3	·5	1·5	1·5	·2	1·8	1·2	·0	2·5
March	·0	·3	1·5	·8	1·6	·0	·0	2·0	·5	·8	·0	4·9	3·	1.1	·0	1·8
April	1·0	·5	1·7	1·9	1·6	·1	2·1	·7	1·4	·5	·8	3·6	15·9	6·5	·0	4·1
May	·5	2·3	·0	4·1	6·0	·7	·0	4·1	6·1	·5	·8	19·7	9·5	1·5	2·6	8·5
June	4·5	16·5	3·1	10·6	10·9	9·1	5·8	12·1	6·9	10·	10·5	37·0	243·3	·9	6·0	20·9
July	14·3	6·9	4·4	10·6	14·1	·9	9·6	15·5	13·6	14·	14·4	36·7	145·3	2·3	19·8	30·1
August	7·4	9·8	4·1	13·5	15·2	·0	4·2	16·6	9·7	12·	3·5	23·5	103·1	1 7	5·0	31·8
September	4·4	4·8	·0	9·0	9·6	·0	2·7	12·5	5·9	6.	2 4	7·1	77·9	2 6	1·8	14·2
October	...	6·8	7·4	4 0	5·4	·0	10·0	6 5	4·9	2·5	7·9	50·2	17·3	15·2	2·3	7·7
November	...	·0	·0	1·0	·0	·0	·0	·0	·0	·5	·0	24·7	·0	15·1	·2	·4
December	...	·4	·0	·3	·0	·0	·0	1 6	·2	2 3	·0	18·0	·0	8 3	·0	·5
Year	33·	48·	23·	57·	69·	11·	36	73	50·	50·	43	230·	613·	57·	38·	123·

	BURDWAN.					PRESIDENCY.		
Place	Bardwan.	Ranigunj.	Bankura.	Suri.	Midnapur.	Calcutta.	Kishnaghur.	Jessor.
Latitude ..	23° 14′	23° 40′	23° 13′	23° 52′	22° 25′	22° 33′	23° 24′	23° 9′
Longitude ...	87° 54′	87° 12′	87° 6′	87° 35′	87° 19′	88° 21′	88° 22′	89° 7′
Height	97	326	...	157	108	18	...	20
No. of Years	13	7	13	10	10	32	10	12
Actual Years.	before 1869	before 1873	before 1869	before 1872	before 1873	before 1869	before 1869	before 1869
January	0·6	·2	·4	·4	·7	·5	·6	·3
February	1·2	·5	1·0	·7	·3	·7	1·0	·5
March	1·5	1·1	1·5	·7	1·8	1·2	·8	1·6
April	1·9	1·1	1·9	·9	1·7	2·2	4·9	3·9
May	4·4	2·3	2·7	2·7	5·9	5·4	8·3	7·7
June	11·2	9·5	10·3	8·3	12·0	11·8	11·8	12·6
July	13·3	14·0	12·5	12·4	11·5	13·3	9·1	10·9
August	11·6	12·1	9·9	12·0	11·1	14·2	9·7	10·3
September	8·9	8·9	8·2	8·6	9·2	10·4	7·9	9·8
October	5·7	3·3	4·2	4·5	6·1	5·3	5·2	6·5
November	0·6	·3	·2	·1	·3	·7	·4	1·0
December	0·6	·0	·1	·2	·0	·2	·2	·0
Year ..	61·	53·	53·	52·	61·	66·	60	65·

	RAJSHAHYE.						COOCH BEHAR.		
Place	Barhampur.	Dinajpur.	Malda.	Bauliah.	Rangpor.	Bograh.	Darjiling.	Rangbi.	Buxa.
Latitude ..	24° 6′	25° 37′	25° 0′	24° 23′	25° 43′	24° 50′	27° 3′	..	26° 48′
Longitude ...	88° 17′	88° 41′	88° 9′	88° 33′	89° 15′	89° 10′	88° 18′	...	89° 30′
Height	65	.	160	110	..	...	6941	5000	1800
No. of Years	.	10	14	10	7	10	10	5–6	3–4
Actual Years.	various. old.	before 1869	before 1869	before 1869	before 1869	before 1870	before 1869	before 1873	before 1873
January	·4	·1	·9	·2	·2	·4	·7	·8	·4
February	·9	·6	1·0	1·4	·3	1·0	1·6	1·5	·8
March	1·2	·6	1·0	1·6	·9	1·0	1·6	2·7	1·5
April	2·4	3·1	1·9	2·7	3·3	5·1	3·6	6·4	7·2
May	4·2	9·7	3·6	6·2	9·9	9·5	7·0	10·5	18·7
June	9·2	18·9	8·6	12·0	23·4	16·4	27·5	31·3	55·5
July	10·2	17·5	9·6	12·9	16·5	20·8	29·4	52·4	63·3
August	9·6	14·4	9·8	8·8	14·0	12·7	29·1	34·1	37·8
September	8·5	14·8	10·1	10·7	11·4	15·6	18·1	27·1	32·9
October	6·1	5·8	4·4	6·3	4·5	6·0	6·6	8·5	17·7
November	·2	·3	·3	·4	·4	1·7	·2	·6	1·0
December	·1	·0	·6	·1	·2	·1	·1	·2	·1
Year ..	53·	86·	52·	63·	85·	90	130·	176·	238·

	DACCA.					ONG.	
Place	Dacca.	Faridpur.	Mymensingh.	Silhet.	Silchar.	Chittagong.	Noakhali.
Latitude ..	23° 43′	23° 35′	24° 49′	24° 55′	24° 49′	22° 21′	22° 58′
Longitude ...	90° 27′	89° 45′	90° 24′	91° 50′	92° 50′	91° 5′	91° 10′
Height	35	...	...	...	89	90	..
No. of Years	10	4	4	14	11	12	13
Actual Years.	before 1869	before 1873	before 1873	before 1869	before 1869	before 1869	before 1869
January	·4	·0	·0	·2	·5	·4	·4
February	·9	·6	·4	1·6	3·5	1·6	·8
March	1·3	1·8	·6	4·9	6·1	1·3	1·6
April	7·5	5·9	3·5	14·4	12·7	5·5	4·3
May	9·8	10·9	8·0	25·0	16·1	9·4	9·1
June	13·9	14·2	21·4	30·9	19·5	22·9	21·3
July	13·4	11·6	14·9	25·9	34·6	22·5	16·6
August	12·3	14·0	15·8	23·6	16·8	23·0	19·6
September	8·2	10·5	10·1	13·5	13·9	13·0	14·9
October	6·3	4·6	5·4	8·7	7·8	5·9	7·9
November	1·1	·1	·0	·8	1·0	2·3	1·8
December	·1	·0	·1	·2	·8	·5	·0
Year ..	75·	74·	80·	150·	123·	108·	98·

II.—Bengal.—Mean Monthly Rainfall for 48 Places for more than 10 years up to 1869 or up to 1873 inclusive.—Continued

	Chittagong.		Bahar.					Monghyr.		Cattak.					Chota Nagpor.			Assam.						
Place … …	Tipperah.	Akyab.	Patna.	Gya.	Arrah.	Chapra.	Champarun.	Monghyr.	Bhagulpur.	Cattak.	False Point.	Puri.	Balasor.	Sambhalpor.	Hazaribagh.	Ranchi.	Porulia.	Goalpara.	Gaohatti.	Tezpor.	Naogong.	Sibsagur.	Shillong.	Cherr
Latitude	23° 27′	20° 8′	25° 37′	24° 42′	25° 32′	25° 48′	…	25° 22′	25° 14′	20° 29′	20° 20′	19° 48′							26° 11′	26° 36′	…	…	25° 32′	25° 16′
Longitude …	91° 5′	92° 57′	85° 8′	85° 2′	84° 46′	84° 50′	…	86° 30′	87° 0′	85° 54′	86° 47′	85° 4							91° 47′	92° 50′	…	…	91° 50′	91° 44′
Height ……	..	21	179	347	191	182	..	160	146	80	19	15							…	…	…	…	4825	4460
No. of Years	10	11	10	10	14	14	10	14	14	12	4	13							19	10	10	12	8	11
Actual Years.	before 1869	before 1869	before 1871	before 1871	before 1869	before 1869	before 1871	before 1869	before 1869	before 1869	before 1873	before 1869	before 1869	before 1871	before 1873	before 1869	before 1873	before 1873	before 1869	before 1869	before 1869	before 1869	before 1873	before 1869
January ……	·8	·4	·5	·9	1·0	·8	·2	·4	·5	·5	·0	·1	1·2	·5	·3	1·0	·2	·3	·7	·7	1·4	1·1	·1	·5
February ……	1·1	·4	·8	·5	·6	·6	·3	·7	·8	·5	·2	1·5	1·3	·6	·4	1·0	·3	·5	1·4	1·0	1·4	2·4	·6	3·6
March	2·3	·5	·3	·7	·7	·5	·9	·4	·5	1·2	1·3	·7	2·2	·6	1·3	1·7	1·2	1·7	1·5	1·9	3·0	3·8	1·5	3·9
April ………	9·3	1·4	·5	·6	·9	·5	·4	·4	1·0	1·4	2·7	1·4	2·3	·4	·4	·6	1·4	5·5	7·3	6·8	5·9	10·1	3·7	30·7
May ….. …	11·5	8·0	1·1	·9	1·2	1·3	1·7	1·7	2·8	1·6	2·6	2·6	4·9	1·1	1·2	1·4	2·3	12·3	10·9	9·5	11·0	11·0	8·6	76·7
June ….. …	20·4	65·2	6·8	7·0	7·7	6·3	8·1	6·0	8·6	9·6	13·7	7·9	12·8	10·0	8·1	6·6	7·7	26·5	13·3	13·8	11·9	15·6	16·0	118·9
July .. …..	17·1	55·8	10·6	12·5	14·4	8·6	10·5	10·4	11·6	11·0	13·7	9·0	8·9	13·1	14·9	10·1	14·8	19·4	13·1	16·5	15·1	14·9	14·1	157·3
August . …	13·8	41·7	7·4	10·1	8·9	7·7	11·2	7·9	11·2	11·2	11·0	12·4	12·6	10·5	13·6	9·0	13·8	11·9	11·9	13·1	13·2	13·9	10·1	80·0
September . .	10·4	24·9	7·3	7·4	10·6	6·0	8·2	7·3	8·4	9·0	10·6	9·6	13·8	7·3	8·4	5·8	5·2	12·2	6·8	7·9	11·2	11·1	16·9	60·5
October ……	7·0	14·7	2·6	3·4	2·7	2·6	3·6	3·7	5·3	5·6	15·3	7·4	7·4	4·4	3·2	3·5	2·4	4·8	3·6	3·1	4·3	4·5	5·2	17·7

III.—Bombay.—Mean Monthly Rainfall for 24 Places during [illegible]

Place	Vingorla.	Dharwar.	Belgaum.	Sawantwari.	Kaladghi.	Kolapur.	Rutnaghiri.	Sholapur.	Sattara.	Malcompet.	Mahableshwar.	Purandhar.	Puna.	Serur.	Bombay.	Nasik.	Dhulia.	Baroda.	Rajkot.	Ahmadabad.	Ahmadnagar.	Kotri.	Mount Abu.	Haiderabad.
Latitude	15° 50′	15° 50′	15° 52′	15° 56′	16° 11′	16° 42′	17° 0′	17° 40′	17° 40′	17° 56′	17° 59′	18° 12′	18° 30′	18° 50′	18° 53′	20° 0′	20° 51′	22° 16′	22° 18′	23° 0′	23° 24′	23° 54′	24° 45′	25° 30′
Longitude	73° 41′	75° 10′	74° 42′	74° 1′	75° 33′	74° 18′	73° 20′	76° 0′	74° 2′	73° 41′	73° 30′	73° 54′	74° 0′	72° 58′	72° 52′	73° 47′	74° 45′	73° 14′	70° 50′	72° 0′	73° 1′	68° 46′	72° 49′	69° 5′
Height	20	2482	2260	...	1750	1797	150	1821	2320	...	4500	4200	1800	1752	14	1843	1000	90	450	320	1900	..	4000	99
No. of Years	4	6	10	2	5	10	9	11	5	6	15	6	18	5	34	4	6	17	4	2	10	5	5	2
Actual Years.	1856–59	1844–47 1859–60	1841–47 1858–59	1848–49	1855–59	1850–59	1844–47 1855–59	1850–60	1855–59	1842–47	1829–43	......	various 1826–60	1854–58	1817–50	1844–47	1853–58	1837–53	1857–60	1858–59	1844–47 1854–59	1845–49	1855–59	1856–57
January	·0	·0	·0	·0	·1	·4	·0	·1	·6	·4	·0	·1	·4	·1	·0	·0	·0	·0	·0	·0	·1	·0	·8	·6
February	·0	·0	·0	·0	·0	·0	·0	·5	·0	·0	·2	·3	·0	·3	·0	·0	·2	·0	·0	·0	·1	·0	·2	·5
March	·0	·6	·8	·0	·1	·6	·0	·3	·1	1·7	·1	·2	·7	·4	·0	·0	·6	·0	·0	·0	·4	·1	·0	·0
April	·4	1·3	3·3	·0	1·5	1·4	·0	1·9	1·0	2·3	1·3	·5	·8	·9	·0	·0	·5	·0	·0	·0	·4	·0	·3	·6
May	16·7	3·7	3·9	2·2	3·8	3·6	3·6	2·3	3·3	2·2	3·3	5·7	3·8	2·9	·0	·0	2·3	·8	·0	·0	3·0	·4	·3	·0
June	29·0	4·6	7·9	32·2	1·6	6·6	31·0	3·7	5·0	48·6	46·5	10·2	6·9	3·7	22·1	6·8	15·3	6·2	4·2	1·8	4·6	1·3	10·3	·0
July	40·9	7·1	15·0	48·3	2·9	11·9	30·5	4·3	13·1	100·2	92·1	23·0	11·1	3·0	24·9	6·7	21·4	14·2	8·3	15·9	2·5	·0	25·9	1·9
August	22·2	3·4	8·4	29·2	2·5	5·1	14·9	6·2	7·1	68·3	72·3	16·3	6·5	1·5	16·8	3·4	9·5	7·4	8·9	7·6	3·3	·0	17·0	14·8
September	6·2	3·0	4·3	16·5	4·1	4·6	9·0	6·8	3·1	33·8	31·3	7·4	4·8	3·9	11·1	2·9	20·9	5·2	5·2	6·0	6·5	·0	9·9	2·1
October	2·9	2·8	5·2	2·6	6·7	4·5	21·9	4·7	4·7	2·4	4·6	6·5	4·5	3·7	1·3	3·3	13·3	·8	·0	·0	3·2	·0	·0	·0
November	·1	3·2	2·4	·3	·6	·7	1·3	1·4	·6	1·7	2·1	·7	3·9	·7	·0	1·5	·0	·0	·0	·0	1·1	·0	·0	·0
December	·0	·1	·0	·0	·4	·1	·0	·5	·6	·1	·0	1·4	·2	·0	·0	·0	·3	·0	·0	·0	·4	·0	·0	·0
Year	118·	35·	51·	131·	24·	39·	111·	33·	39·	262·	254·	72·	44·	21·	76·	29·	84·	35·	27·	31·	26·	2·	65·	21·

III.—BOMBAY.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.

Place	SPECIAL OBSERVATORIES.								
	Belgaum.	Puna.	Bombay.	Karachi.	Deesa.	Ahmadabad.	Ahmadnagar.	Ahmodi.	Alibagh.
Latitude	15° 52′	18° 30′	18° 53′	24° 51′	25° 14′	23° 0′	23° 34′	21° 59′	18° 40′
Longitude	74° 42′	74° 0′	72° 52′	67° 2′	72° 5′	72° 0′	73° 1′	72° 53′	72° 58′
Height	2260	1800	14	27	400	320	1900	...	..
No. of Years.	10	10	10	10	10	7	8	6	7
January	·00	·01	·04	·85	·22	·0	·4	·0	[illegible]
February	·01	·10	·01	·19	·18	·0	·0	·0	[illegible]
March	·51	·34	·00	·26	·25	·0	·3	·0	[illegible]
April	1·92	·44	·01	·01	·00	·0	·5	·0	·0
May	1·83	1·01	·08	·15	·03	·0	·9	0	·0
June	12·11	7·45	17·24	·17	2·82	3·9	5·2	5·9	18·6
July	14·82	5·74	24·06	2·72	8·13	7·8	2·6	7·1	24·4
August	12·79	5·95	17·94	2·74	10·31	13·6	4·6	13·1	16·9
September	2·01	4·29	9·47	1·03	2·40	2·4	5·6	2·6	9·2
October	4·56	6·77	2·42	·19	1·63	·0	3·8	·6	·3
November	·28	·48	·23	·05	·07	·0	·5	·0	·0
December	·62	·35	·01	·29	·00	·0	·0	·0	·0
Mean Year	51·44	32·99	71·53	8·65	25·93	27	24	29	69
Min. Year	42·98	22·34	45·61	2·41	15·93	18	17	15	57
Max. Year	64·69	57·79	91·66	28·45	38·94	46	37	42	78

Place	Amalner.	Anand.	Ankola.	Asry.	Badami.	Baglan.	Bagalkot.	Bajaiwadi.	Balsar.	Baramati.		
Latitude	28° 3′	...	...	...	15° 55′	..	16° 12′	...	20° 36′	18° 10′	21° 4′	22° 16′
Longitude	75° 1′	...	...	.	45′	...	75° 25′	...	72° 58′	74° 30′	73° 10′	73° 14′
Height	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	...	...	48	...	...	90
No. of Years.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3	4	8	2	8	7
January	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
February	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
March	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0	·0	·2
June	4·0	5·5	5·2	3·6	2·2	3·6	3·1	1·8	17·4	4·2	8·0	6·3
July	8·4	7·3	5·3	8·9	3·5	3·9	2·9	4·4	23·3	·8	12·0	8·1
August	6·7	10·9	5·1	4·0	1·6	3·4	1·7	4·0	18·5	3·7	22·7	13·8
September	4·5	·0	3·3	2·3	5·5	2·8	5·8	3·4	9·0	2·2	5·6	2·4
October	1·0	1·2	3·5	4·3	7·1	1·4	4·7	3·6	·1	·0	1·0	0·3
November	·0	·0	·2	·1	·0	·1	·0	·9	·0	·0	·0	·1
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	25	24	23	17	20	15	18	18	68	11	49	31
Min. Year	15	16	15	12	14	9	15	12	47	8	29	21
Max. Year	57	34	25	23	25	25	23	29	87	14	62	45

N.B.—The Latitudes and Longitudes given for these places in the Bombay Presidency have been taken from a map, as none were given with the [illegible]

Place …………	Barsi.	Bassein.	Bedi.	Belgaum.	Bejapur.	Belzi	Bhargaum.	Bhewandi.	Bhaunagar.	Brahmanwade.	Broach.	Borsad.	Byculla.	Carwar	Callian.	Callian.	Chaligaum.	Chandor.	Chikli.	Chikodi.	Chiplon.	Chopra.	Colaba.
Latitude ………	18° 16'	19° 20'	…	15° 52'	16° 50'	…	..	19° 19'	21° 45'	.	21° 42'	…	18° 57'	14° 50'	…	.	20° 40'	20° 20'	22° 19'	..	17° 30'	21° 14'	18° 53'
Longitude…………	75° 46'	72° 52'	.	74° 42'	75° 40'	…	…	73° 9'	72° 10'	..	73° 2'	…	72° 52'	74° 15'	…	..	75° 5'	74° 14'	74° 41'	..	73° 36'	75° 27'	72° 52'
Height . . ………	1682	11	.	2260	…	…	…	…	…	…	56	…	13	…	…	26	1138	1806	…	…	…	..	14
No. of Years …..	3	7	7	5	2	3	7	3	8	5	7	8	7	8	3	3	8	8	8	7	4	8	7
January ……… ..	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February .. . … …	·0	·0	0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March ……… …	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·4	·0	·0	·0	·0	·0	·0	·0	·0	·0
May …. ….	·0	·0	1·0	1·9	·0	1·0	·0	·0	·0	·0	·0	·0	·0	1·1	·0	0	·0	·1	·0	·6	·2	·0	·2
June …. …………	5·0	16·2	11·8	10·6	5·4	3·0	4·1	13·7	5·3	4·5	7·0	4·5	20·7	34·4	16·7	16·7	5·2	3·9	16·7	5·2	37·3	3·6	15·5
July …. ………	3·3	20·3	11·2	9·6	1·2	1·3	6·7	24·2	4·8	4·5	11·8	9·0	26·2	36·9	31·2	31·2	5·4	4·7	26·9	4·1	42·4	6·8	24·4
August . ….. ……	4·4	17·3	8·1	7·1	4·0	2·0	7·2	16·7	10·4	7·2	11·4	14·7	20·4	23·7	27·9	27·9	5·7	5·4	20·9	3·4	36·9	4·9	18·1
September …….	8·4	8·7	1·7	2·6	8·2	5·4	4·4	7·5	1·9	2·8	3·8	3·4	8·7	9·0	6·4	6·4	5·6	3·8	6·9	2·7	9·5	2·1	8·7
October . .. …….	3·0	·7	1·3	1·7	2·8	1·8	·9	2·7	1·1	·0	·4	·1	·7	3·4	3·2	3·2	·9	2·8	5·9	1·4	4·3	·3	·8
November ……. .	·1	·0	·1	·0	·0	·1	·0	·2	·0	·0	·0	·0	·0	1·1	·0	·0	·0	·2	·0	·0	·3	·0	·0
December.. . ……	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year …. ..	24	63	36	34	22	13	23	65	24	19	34	32	77	110	85	85	23	21	77	17	131	20	68
Min. Year .. ……	19	54	27	32	17	.	19	7	9	17	21	21	56	80	81	81	13	14	13	53	119	14	45
Max. Year ………	27	78	57	50	43	18	34	96	42	22	51	50	80	145	91	91	27	28	104	26	145	32	81

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Cumpta.	Dahnu.	Dahwudi.	Dapuli.	Dewarhobli.	Dohud.	Dumbul.	Dindor.	Dhalka.	Dharwar.	Dhundunka.
Latitude	14° 25′	19° 55′	...	17° 48′	...	..	15° 18′	20° 1′	...	15° 28′	...
Longitude	74° 20′	72° 40′	..	73° 16′	...	..	75° 50′	73° 50′	...	75° 54′	..
Height	...	..	..	...	..	...	...	...	...	...	..
No. of Years	8	7	7	8	7	8	7	8	6	4	6
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	[illegible]
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	[illegible]
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·5	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·5	·0	·0	·0	·0	·0	·0	·0	·0	3·4	·0
June	31·8	15·5	3·6	30·7	5·7	5·5	2·7	4·6	2·5	7·2	3·6
July	44·7	19·3	1·4	39·4	6·7	10·9	1·1	7·6	7·3	4·8	5·7
August	23·5	19·8	2·1	31·7	5·4	8·2	1·8	8·9	11·1	4·8	11·3
September	7·6	7·6	3·6	11·3	2·1	2·8	2·6	3·2	1·7	2·6	2·4
October	3·7	·8	3·8	3·3	1·3	·7	·9	3·7	·0	2·5	·1
November	·3	·0	·0	·0	·0	·0	·0	·2	·0	·0	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	113	63	15	117	21	28	9	27	23	25	23
Min. Year	11	43	9	91	16	24	6	21	13	18	11
Max. Year	163	79	20	145	26	35	18	34	31	31	33

Place	Dhulia.	Egutpura.	Erandol.	Esplanade.	Fort St. George.	Ghodnuddi.	Godai.	Godra.	Gogo.	Gohagar.	Gokak.	Gulaj.
Latitude	20° 54′	19° 43′	20° 56′	18° 57′	18° 57′	...	..	22° 45′	21° 39′	...	16° 10′	15° 26′
Longitude	74° 45′	73° 34′	75° 19′	72° 52′	72° 52′	...	...	73° 36′	72° 15′	..	74° 53′	75° 43′
Height				34	34	...	...	...	...	...	...	...
No. of Years	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	7	8	7	8	7	7
January	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
February	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·5	·0
June	4·0	23·9	5·1	19·6	19·9	4·6	4·7	5·9	4·9	22·2	3·4	3·0
July	3·5	40·3	6·0	25·4	26·2	1·0	3·9	7·5	6·1	31·7	1·5	1·4
August	4·1	42·7	7·4	19·4	19·3	2·5	6·1	18·9	6·5	17·6	1·9	3·2
September	2·3	13·6	5·1	9·4	8·8	3·2	1·3	2·3	2·5	8·1	2·2	2·7
October	·7	4·9	·5	·6	·6	·6	·4	·1	·0	2·7	1·8	1·3
November	·1	·0	·0	·0	·0	·1	·1	·0	·0	·7	·1	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	15	126	22	74	75	12	10	35	20	83	11	12
Min. Year	9	105	16	57	62	10	11	28	15	56	4	7
Max. Year	27	161	34	84	87	14	13	42	30	141	18	21

III.—BOMBAY.—*Mean Monthly Rainfall of 205 Places between* [illegible]

Place	Gukol.	Halial.	Halole.	Hangal.	Hansot.	Hirvat.	Honor.	Hobli.	Hungund.	Indapor.	Indi.	Jamner.	Jhaloda.	Kadkalay.	Kaira.	Kalole.	Karjat.	Karjat.	Kallawadi.	Karangaum.	Kharepatan.	Khed.	Khed.
Latitude	14° 50′	15° 20′	...	14° 50′	21° 32′	...	14° 10′	15° 20′	16° 3′	18° 10′	17° 10′	20° 48′	23° 5′	...	22° 45′	...	...	...	...	...	...	18° 51′	18° 23′
Longitude	75° 42′	74° 40′	...	75° 5′	72° 50′	...	74° 25′	75° 15′	76° 9′	75° 5′	75° 57′	75° 44′	74° 3′	...	72° 41′	...	...	...	...	...	...	73° 55′	73° 59′
Height	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
No. of Years	·6	·8	3	7	4	5	7	7	3	7	3	8	5	7	8	3	8	7	8	8	8	7	6
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	1·2	·0	·0	·0	·0	·6	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	1·3	·0	·0	·0	·0	1·7	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0
June	2·1	14·9	3·3	6·1	2·4	4·2	34·3	4·7	2·4	3·3	·3	4·2	1·8	14·4	4·1	4·3	3·1	21·6	11·5	3·6	34·8	5·1	33·3
July	1·8	20·7	8·5	8·6	9·6	3·6	32·2	3·9	4·2	1·2	6·1	7·0	9·8	22·5	9·5	10·1	1·8	39·3	18·3	2·0	43·1	3·5	41·7
August	2·4	19·2	20·0	6·0	16·7	4·5	26·0	3·3	2·2	1·0	3·1	6·8	13·4	23·3	11·1	17·3	2·1	39·7	15·4	3·9	29·2	4·6	30·8
September	3·0	4·2	2·0	2·0	2·1	1·8	15·1	2·2	6·4	1·8	3·9	3·3	2·1	3·9	2·1	2·1	5·4	10·7	4·4	3·8	10·9	2·3	9·1
October	1·8	7·3	·1	1·3	1·2	·0	3·2	1·1	3·6	·0	8·0	·4	·8	·0	·1	·8	1·3	1·3	1·1	3·1	4·5	·4	3·9
November	·0	1·0	·0	·0	·0	·0	·6	·0	·0	·0	3·3	·0	·0	·0	·0	·0	·1	·0	·0	·5	·4	·0	·4
December	·0	·2	·0	·0	·0	·0	·2	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	11	70	34	24	32	14	114	15	19	7	25	22	28	64	27	35	14	113	51	17	123	16	119
Min. Year	7	28	28	22	23	7	8	8	16	3	16	17	26	45	26	27	11	85	26	12	102	11	100
Max. Year	18	132	37	29	48	21	175	22	24	11	29	25	30	93	33	39	21	136	70	28	143	26	133

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Khandalla.	Karadh.	Kurnalla.	Kulladghi.	Kunkowli.	Kuppurwang.	Kurjugi.	Kolhapur.	Koregaum.	Lanza.	Lohara.	Mahad.	Mahim.	Malshiras.	Malcolmpet.	Malwan.	Malligaum.	Malunda.	Mangaon.	Mandvee.	Manikwara.	Matur.	Medhai.
Latitude	18° 48′	17° 15′	18° 25′	16° 10′	...	...	...	16° 41′	17° 41′	...	20° 42′	...	19° 40′	...	17° 56′	...	20° 32′	...	15° 20′	22° 51′	21° 20′	...	...
Longitude	73° 26′	74° 10′	75° 15′	75° 25′	...	...	...	74° 10′	74° 25′	...	75° 32′	...	72° 47′	...	73° 41′	...	74° 30′	...	73° 55′	69° 26′	70° 40′	...	...
Height	1793	...	...	...	...	...	...	...	...	...	..	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	...	...	...	...	...	...	...
No. of Years	2	8	7	4	7	8	7	8	8	3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	6	2	8	2	5	8
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·2	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·2	·0	·0	·0	1·8	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0
May	·1	·0	·1	1·1	·2	·0	·0	2·1	·0	·3	·0	·0	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0	·2
June	7·4	6·5	3·9	3·2	31·0	4·9	3·1	8·7	6·6	26·5	5·5	27·9	16·1	2·4	16·7	18·7	4·2	4·5	30·9	9·4	2·2	5·2	16·9
July	4·9	6·5	1·9	1·4	41·7	8·8	3·5	9·1	6·1	47·3	9·1	45·6	22·7	1·5	95·1	32·1	3·2	11·1	36·6	15·1	6·6	9·9	21·0
August	2·4	5·9	2·3	3·1	26·7	10·8	3·1	7·9	6·1	25·0	8·6	37·7	20·2	2·4	86·1	11·7	3·9	9·9	30·6	17·8	8·0	6·8	20·0
September	1·2	1·9	4·3	3·2	10·2	3·1	2·0	2·9	2·4	11·1	5·0	12·6	7·1	4·9	55·9	5·6	5·7	2·0	11·4	3·9	3·1	5·1	2·7
October	5·1	5·5	2·1	3·2	4·0	·1	2·0	5·1	4·0	4·2	·5	1·1	1·8	3·9	6·6	4·8	1·4	·0	8·2	1·4	2·1	·3	3·6
November	·1	·2	1·0	·1	·2	·0	·0	·3	·2	·6	·0	·0	·0	·1	·4	·2	·0	·0	·0	·0	·1	·0	·3
December	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	21	26	15	16	114	28	14	38	25	115	29	125	68	15	260	73	18	28	118	48	22	27	65
Min. Year	15	20	4	7	100	15	8	29	22	100	19	98	59	9	214	58	12	14	97	36	22	25	52
Max. Year	27	32	30	27	140	52	20	46	36	128	37	171	81	28	312	86	26	42	138	57	22	32	81

Place	Mehmand	Murikot.	Moolshi.	Mokhada.	Mohole.	Murbad.	Mundnag	Mudibah	Nariad.	Nasik.	Nasirabad.	Niphar.	Newasa.	Nundoor	Nuppur.	Nurgoond	Nuvulgoo	Oomburg	Ooljar.	Oorun.	Pabal.	Padshap	Palitana.
Latitude	18° 42′	15° 16′	...	...	17° 48′	19° 17′	..	16° 17′	..	20° 0′	20° 59′	20° 5′	...	21° 22′	...	15° 43′	...	...	...	..	..	16° 5′	21° 30′
Longitude...........	76° 18′	78° 8′	..	.	75° 42′	73° 30′	..	76° 5′	..	73° 47′	75° 37′	73° 45′	...	74° 12′	...	75° 27′	...	...	...	...	..	74° 46′	71° 40′
Height	5	7	7	7	1493	.	...	...	...	1843	..	...	...	..	..	...	...	...	...	..	...	..	..
No. of Years..	...	...	...	...	4	7	7	5	8	8	8	8	8	8	3	7	7	2	8	2	5	5	8
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
June	1·9	5·8	12·2	17·8	4·2	15·2	33·6	2·8	4·7	4·5	4·4	3·1	3·4	4·3	5·5	3·2	4·4	5·7	6·5	10·2	4·4	10·5	·0
July	5·5	6·3	15·8	30·7	3·7	26·7	46·0	2·1	10·7	6·1	8·5	2·4	3·0	6·1	10·6	1·6	2·8	20·8	8·5	32·6	2·6	12·1	12·4
August .. .	11·9	4·1	14·6	31·2	3·3	28·8	32·7	4·7	11·3	6·3	7·2	4·5	2·9	8·0	5·5	2·5	3·3	22·3	9·9	20·6	3·9	9·1	13·5
September	1·4	2·3	3·2	9·6	2·6	8·1	11·6	3·6	1·9	3·6	4·8	2·4	4·7	2·2	2·3	2·5	4·0	5·1	8·6	9·5	3·0	1·8	2·5
October...	·0	1·8	·1	1·6	5·3	·9	3·9	2·4	·1	3·9	·4	3·7	1·5	·2	·0	1·7	1·3	3·0	·5	3·1	·0	·0	1·1
November.	·0	0	·0	·0	·1	·0	·7	·5	·0	·3	·0	·1	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
December. ..	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	21	20	46	91	19	71	128	16	29	25	25	16	16	21	24	12	16	57	28	76	14	34	30
Min. Year	15	9	27	74	13	65	109	8	17	16	17	11	6	13	19	6	6	40	16	73	7	30	20
Max. Year	31	26	55	108	30	92	136	23	40	32	36	20	23	26	31	19	30	74	49	80	18	40	42

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Pamkher.	Panwell.	Pandarpur.	Paragud.	Pardee.	Parneir.	Parentay.	Patun.	Patus.	Peint.
Latitude	...	18° 58′	17° 40′	...	...	19° 0′	...	...	18° 28′	...
Longitude	...	73° 12′	75° 24′	...	...	74° 29′	...	...	74° 25′	...
Height	...	...	...	...	...	...	...	...	...	...
No. of Years	8	7	4	7	8	8	6	7	7	7
January	·2	·0	·0	·0	·0	·1	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·0	·0	·1	·0	·1	·0	·3	·1	·0
June	6·6	22·8	3·7	4·5	16·2	5·1	3·7	13·1	4·2	21·5
July	3·4	36·4	2·2	2·5	24·6	2·4	7·8	16·7	·8	32·8
August	5·5	31·8	3·3	2·2	21·9	4·0	14·8	18·6	2·0	23·2
September	6·9	10·6	4·2	2·8	8·2	4·3	2·9	3·1	3·7	7·7
October	3·4	1·3	3·4	1·7	1·3	4·6	·0	3·4	·1	·3
November	·2	·0	·0	·0	·0	·3	·0	·2	·0	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	26	103	17	15	72	21	29	55	11	85
Min. Year	16	90	7	6	48	14	24	42	7	68
Max. Year	38	112	26	27	95	28	34	84	20	92

Place	Peint.	Pimpulner.	Puna.	Rahuri.	Rajkot.	Ranibedor.	Raveri.	Rewarkanta.	Rohey.	Rojapur.	Ratchal.	Rutnagheri.	Sadra.
Latitude	20° 17′	20° 55′	18° 31′	19° 23′	22° 18′	14° 37′	21° 14′	...	...	...	...	16° 58′	...
Longitude	73° 31′	74° 4′	73° 53′	74° 40′	70° 50′	75° 41′	76° 14′	...	...	...	...	73° 15′	...
Height	...	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	...	...	...	...	...	...
No. of Years	4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5	7	4	7	8	8
January	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
February	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·0	·0	·0	·0	·0	·0
March	·0	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·2	·0	·0	·0	·0	·0	·0	·0	·0	·3	·0
May	·1	·0	·6	·1	·1	·0	·0	·0	·0	·2	·0	·3	·1
June	14·1	3·6	5·2	4·8	6·8	·5	5·4	4·3	25·6	31·2	4·5	30·6	4·6
July	35·8	5·7	4·6	2·2	8·9	2·2	6·0	9·7	36·2	40·9	4·2	35·2	10·5
August	37·0	6·7	5·4	3·2	10·1	2·1	7·7	14·8	29·9	34·3	3·5	23·2	14·1
September	9·1	2·1	3·6	5·8	3·1	2·0	4·7	2·2	9·8	7·9	8·6	11·4	3·3
October	4·1	1·3	7·1	2·9	1·5	1·8	·6	·1	1·0	3·6	1·7	2·6	·7
November	·0	·0	·2	·2	·0	·1	·2	·0	·0	·4	·0	·6	·2
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1
Mean Year	100	19	27	19	31	12	24	31	101	119	22	104	34
Min. Year	81	13	19	14	20	7	11	26	89	106	11	90	22
Max. Year	121	22	47	24	44	21	35	41	110	131	67	123	55

Place	Singola.	Sattara.	Sawurgaon.	Sawantwari.	Sanand.	Sassoor.	Shiraloy.	Shahapur.	Sheogaum.	Shrigonda.	Sheagaum.	Shindgi.	Sholapor.	Sinnar.	Sirey.	Soopa.	Soopa.	Soopuree.	Siddopur.	Sungmeshwar	Surat.	Sungamner.	Sultanpur.
Latitude	17° 26′	17° 45′	20° 7′	15° 56′	...	18° 20′	...	...	...	...	...	...	17° 40′	19° 51′	...	18° 20′	15° 16′	...	14° 21′	17° 8′	21° 10′	19° 35′	21° 45′
Longitude	75° 16′	74° 4′	74° 25′	74° 1′	...	74° 3′	...	...	...	...	...	...	76° 0′	74° 0′	...	74° 26′	74° 35′	...	74° 58′	73° 35′	72° 52′	74° 14′	74° 20′
Height	...	...	...	...	...	...	...	...	...	...	...	...	1495	...	...	...	1250	...	...	...	59	...	...
No. of Years	4	2	8	8	2	7	3	7	8	2	7	5	8	8	8	5	8	2	7	4	8	8	8
January	·0	·0	·1	·0	·0	·0	·0	·0	·2	·2	·0	·0	·2	·0	·2	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·2	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0	·0	·3	·0	·7	·0	·1	·0	·9	·0	·0	·0	·0
April	·0	·0	·0	·5	·0	·0	·0	·0	·0	·0	·0	·0	·5	·0	1·1	1·1	·0	·0	1·1	·0	·0	·0	·0
May	·0	·1	·0	·4	·0	·0	·4	·0	·0	·2	·0	·0	·8	·0	1·6	·0	·9	·0	1·1	·1	·0	·3	·0
June	3·2	9·5	3·9	40·5	2·4	2·1	8·4	22·8	3·6	7·3	4·8	·1	3·8	4·0	24·1	1·8	14·4	2·4	27·4	31·9	9·7	4·4	4·1
July	2·1	10·2	3·0	46·9	8·3	1·0	6·5	39·7	4·1	2·1	4·5	4·9	4·5	3·4	35·3	·9	40·9	10·6	39·5	50·9	9·9	2·0	6·5
August	2·8	10·8	4·4	33·6	17·6	1·8	7·6	32·3	5·5	2·4	3·2	3·1	5·5	4·8	27·0	1·0	30·8	16·1	27·0	41·2	12·2	3·0	8·8
September	3·8	2·1	2·8	11·0	5·0	1·1	1·5	14·2	5·9	4·0	1·2	3·7	5·5	2·6	16·1	1·9	7·6	5·0	7·5	9·3	3·6	3·5	2·4
October	1·6	4·4	2·7	4·8	·2	·2	6·4	1·8	1·6	5·4	1·9	5·3	2·5	5·5	6·5	·0	5·6	·2	7·0	3·8	1·7	4·4	1·4
November	·0	·6	·6	·6	·0	0	·0	·0	·0	·6	·0	3·3	·4	·4	·5	·0	·8	·0	·3	·7	·1	·7	·0
December	·0	·0	·0	·6	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0	·1	·0
Mean Year	14	38	18	139	33	6	31	111	21	22	16	20	24	21	113	5	101	34	112	138	37	18	23
Min. Year	7	29	10	97	23	2	27	96	14	16	12	10	19	16	89	4	74	26	27	125	20	11	17
Max. Year	22	48	22	176	44	17	35	150	27	28	21	27	28	26	189	8	146	43	144	152	52	29	29

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1866 inclusive.—*Continued.*

Place	Sumpgure.	Sowda.	Thalner.	Thaura.	Tanna.	Tasgaum.	Tooneer.	Tumbooner.	Unkleser.	Uthnee.	Veerangaon.	Vachoor.	Vingorla.	Virdale.	Viery.	Wadoor.	Wadwan.	Waee.	Wagra.	Walore.	Warra.	Wurrungaum.	
Latitude	...	21° 8'	21° 15'	...	...	17° 2'	...	...	...	...	23° 71'	...	15° 50'	...	...	...	22° 40'	19° 50'	...	...	...	...	14° 58'
Longitude	...	76° 0'	75° 6'	...	...	74° 40'	...	...	...	...	72° 0'	...	73° 41'	...	...	...	71° 30'	74° 12'	...	...	...	...	74° 46'
Height	...	708	...	...	...	...	...	...	65	...	...							...	...	...	...	843	...
No. of Years	7	8	8	8	7	6	8	7	6	7	6							8	7	4	2	7	8
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1							·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1							·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·3	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	1·1
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·2	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·6
May	·8	·0	·0	·0	·1	·2	·0	·0	·0	·0	·0	·5	1·3	·0	·1	·0	·0	·0	·0	·0	·0	·0	11·2
June	5·7	5·1	4·4	4·9	23·0	4·1	4·5	4·0	5·0	3·5	4·2	3·1	39·2	4·3	3·8	4·1	2·0	7·3	3·7	6·5	15·1	4·4	20·6
July	3·9	6·9	6·6	9·2	31·7	2·4	5·0	8·6	12·3	1·1	6·0	3·5	39·4	4·6	3·1	2·2	3·3	6·6	8·5	16·2	31·8	7·6	29·8
August	5·2	5·9	6·0	10·9	16·6	3·5	7·8	8·6	14·1	2·7	9·2	4·1	0·3	5·6	4·8	3·8	13·3	6·0	9·3	25·8	29·9	6·3	22·4
September	2·6	3·4	2·2	3·2	9·5	3·3	2·0	5·6	2·9	3·2	3·4	2·1	7·3	2·2	3·0	3·5	4·8	2·3	3·2	5·1	6·8	4·1	4·6
October	1·2	·5	·5	·2	·0	3·5	1·0	·3	·4	1·1	·0	3·1	3·3	·7	6·0	3·0	1·2	4·9	·3	3·1	4·2	·5	4·6
November	·0	·0	·1	·0	·0	·1	·2	·0	·0	·0	·0	·2	·3	·1	·1	·1	·0	·1	·0	·0	·0	·0	·6
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1
Mean Year	16	28	20	28	91	17	21	27	35	12	23	17	111	17	21	17	25	27	25	57	88	23	95
Min. Year	7	14	10	18	76	...	16	13	21	6	14	12	108	12	11	9	11	18	10	41	84	17	63
Max. Year	24	30	31	46	101	..	25	36	45	20	29	24	116	23	38	30	38	33	32	73	92	38	190

	NORTH-WEST PROVINCES.													PANJAB.								
Place	Banaras.	Gharipur.	Azimgarh.	Hamirpur.	Kherwara.	Etawah.	Gorakpur.	Fattehgarh.	Agra.	Mirut.	Rurkhi.	Landaur.	Naini Tal.	Delhi.	Ambala.	Mian Mir.	Firozpur.	Ludhiana.	Panjar.	Miri.	Rawalpindi.	Peshawar.
Latitude	25° 18′	25° 49′	26° 3′	26° 7′	26° 42′	26° 45′	26° 46′	27° 23′	27° 10′	28° 55′	29° 53′	30° 27′	29° 20′	28° 31′	30° 23′	30° 34′	30° 57′	30° 55′	31° 40′	32° 37′	33° 4′	34° 20′
Longitude	83° 3′	80° 48′	83° 13′	79° 47′	79° 12′	79° 3′	83° 18′	79° 30′	78° 5′	77° 40′	77° 57′	78° 10′	79° 31′	77° 13′	76° 44′	74° 4′	74° 41′	75° 54′	74° 45′	70° 30′	73° 5′	71° 29′
Height	270	...	...	...	1200	...	255	600	551	...	880	7000	6463	...	1050	1128	710	900	850	...	1700	1056
No. of Years	2	4	1	1	5	1	1	2	1	1	1	10	9	1	2	3	5	5	3	1	1	6
Actual Years.	1858–59	1856–59	1851	1851	1854–58	1851	1851	1848–49	1851	1859	1860	1850–59	1846–54	1851	1851–52	1857–59	1855–59	1857–59	1857–59	1848	1859	1855–60
January	1·2	·8	3·8	1·5	·0	2·8	3·6	·9	1·2	·2	·0	·6	4·6	2·6	·7	1·2	1·0	2·5	1·1	·0	·0	1·2
February	·1	1·7	1·1	·8	·0	·6	·2	·0	1·1	·2	·0	·6	2·4	·3	·0	1·4	·2	1·1	2·0	·0	·6	1·8
March	·1	·4	·6	·0	·0	·0	·2	·0	·0	1·8	·0	·3	1·7	2·6	7·1	·4	·6	·1	·7	·0	·0	1·1
April	·3	·4	·8	·3	·0	·1	·1	·0	·2	2·6	·0	1·9	·9	1·0	·9	·5	·9	·8	5·2	·0	·0	·8
May	·0	·5	·0	·0	·0	·3	1·4	·0	·7	·0	·5	1·5	2·8	·1	2·8	·4	·1	·5	1·2	2·8	·0	·4
June	6·3	7·4	4·5	6·7	4·0	·9	15·6	2·7	·3	3·6	1·5	11·9	14·7	·4	1·3	1·4	1·1	1·6	8·4	10·9	4·1	·3
July	7·1	8·6	8·7	12·6	12·3	11·4	14·1	2·5	9·8	1·5	4·0	31·5	23·4	11·6	7·3	5·5	6·5	2·9	22·7	22·9	4·8	2·1
August	5·6	14·4	3·8	10·5	3·9	9·2	5·0	5·4	10·0	5·9	3·0	32·3	28·8	6·5	3·1	3·6	1·3	4·3	10·5	6·2	5·1	3·3
September	9·7	8·2	9·5	4·8	6·5	6·2	5·9	10·7	4·0	1·4	1·0	10·3	12·6	·0	·0	1·9	1·9	2·3	4·2	2·3	5·6	1·7
October	3·8	4·2	7·2	·7	·8	·3	11·6	2·9	·6	·0	·3	·9	2·7	·0	·0	·0	·0	·0	·0	2·8	·0	·1
November	·0	·3	·0	·0	·0	·0	·0	·0	·0	·0	·0	·6	·8	·0	2·3	·0	·0	·2	·6	1·9	·0	·3
December	·0	·4	·0	·0	·0	·1	·0	·3	·0	·8	·0	·3	·2	·0	·1	·0	·0	·2	·6	·0	·0	·6
Year	37	47	40	38	28	32	62	25	28	18	10	93	96	25	26	16	14	17	57	50	18	14

IV.—North-West Provinces and Oudh.—Mean Monthly Rainfall between 1867 and 1872 inclusive.

Place	Chakrata.	Ranikhet.	Dehra.	Rurkhi.	Mirat.	Bareli.	Agra.	Fattehgarh.	Lakhnau	Allahabad.	Banaras.	Gorakpur.	Jhansi.	Ajmir.	Morar.	Nagode.	Naugong.	Caunpur.	Sitapur.	Fyzabad.	Naini Tal.
Latitude	30 40	29 38	30 20	29 53	29 41	28 21	27 0	27 23	26 50	25 26	25	[illegible]	[illegible]	26 27	26	35	..	26 37	.	26 48	29 23
Longitude	77 55	79 29	78 8	77 54	77 41	79 27	78 0	79 48	81 0	81 52	83					7	...	80 20		82 10	79 31
Height	7056	6143	2232	880	739	570	551	503	364	289							...	..	..	..	6409
No. of Years	4	2	5	5	5	5	5	5	5	5							1	1	2	2	5
Actual Years.	1869-72	1871-72	1868-72	1868-72	1868-72	1868-72	1868-72	1868-72	1868-72	1868-72	1868-72	1868-72	1868-72	1868-72	1869	1867-69	1869	1866	1868-69	1868-69	1865-69
January	·6	2·4	1·9	2·2	1·7	1·5	·9	·9	·8	·4	·8	1·1	2·0	·1	·3	·9	·1	·0	·8	·4	6·5
February	1·7	3·7	3·0	2·3	1·2	·7	[illegible]	·3	·4	·0	·3	·6	·3	·1	·1	·5	·2	·8	·7	·5	5·1
March	6·7	·2	3·4	1·7	1·9	1·6	1·3	·7	·8	·6	·5	·2	·5	·4	1·0	·6	·5	·0	·4	·6	5·5
April	1·3	1·7	·9	·4	·8	·5	·4	·2	·4	·7	·6	·2	·1	·2	·1	·2	·3	·0	·7	·0	2·4
May	2·1	3·6	1·4	1·1	1·6	1·4	1·3	·9	1·5	·4	·4	2·4	·4	·6	·2	·3	·0	·0	·9	1·2	2·6
June	8·6	9·6	10·3	6·0	4·9	8·4	2·7	3·7	5·2	8·0	6·2	4·4	3·2	2·8	·1	11·1	·6	3·2	1·5	4·1	15·3
July	17·1	10·9	28·7	13·9	8·9	13·1	8·4	12·4	14·5	14·7	14·0	16·5	13·3	6·6	11·0	13·8	33·2	10·9	6·0	14·1	19·3
August	12·1	12·8	23·9	11·0	5·3	6·1	5·7	6·9	11·1	11·6	8·7	13·2	8·8	6·5	3·5	7·7	6·9	8·9	6·5	3·3	22·3
September	6·2	6·3	13·4	5·1	3·0	8·9	4·3	4·8	12·4	11·8	8·1	11·7	5·2	4·5	8·0	12·8	8·9	4·1	3·9	12·8	9·2
October	·7	0	1·3	·9	·0	2·2	·0	0	3·2	12·9	5·1	6·8	1·3	·1	4·8	6·0	8·2	·0	4·5	6·3	4·4

Place	Delhi.	Gurgaon.	Karnal.	Hisar.	Rohtak.	Sirsa.	Ambala.	Ludiana.	Simla.	Jhallandar.	Hoshyarpur.	Kangra.	Amritsar.	Sialkot.	Gurdaspur.	Lahor.	Firozpor.	Gujranwala.	Rawalpindi.	Jhelam.	Guzrat.	Shahpur.	Multan.	Jhang.
Latitude . ..	28° 31′	28° 28′	29° 42′	29° 16′	28° 53′	...	30° 23′	30° 55′	31° 8′	31° 20′	31° 35′	32° 8′	31° 40′	32° 29′		31° 34′	30° 57′	32° 10′	33° 4′	32° 58′	..	...	30° 12′	.
Longitude ..	77° 13′	76° 58′	76° 55′	75° 45′	76° 29′	. .	76° 44′	75° 54′	77° 8′	75° 26′	75° 50′	76° 15′	74° 50′	74° 35′		74° 21′	74° 41′	74° 10′	73° 5′	73° 42′	...	...	71° 25′	...
Height . .	.	.		...	...	.	1050	900	.	..	.	.	.	900	.	1000	720	..	1700	..	...	...	...	..
No. of Years	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
January	·8	1·9	1·0	5	·2	·5	·8	1·0	1·0	·9	1·0	2·7	1·0	1·3	1·0	·5	·5	1·0	1·9	·7	·9	·1	·3	·3
February . .	·5	·3	1·1	·4	·6	·5	2·0	1·0	1·4	1·3	1·4	5·6	1·2	1·9	2·9	1·0	1·0	·8	1·9	1·4	1·6	·6	·1	·5
March	·2	·6	1·6	·7	·8	·7	1·5	2·0	4·0	1·4	1·8	4·9	1·2	2·9	2·6	1·2	1·6	2·0	3·0	1·6	4·1	1·0	9	2·0
April	·5	·3	·4	·2	·4	·3	1·2	·7	2·7	·3	·5	1·7	·4	1·8	·5	·4	·9	1·1	1·9	·8	1·2	·5	1·1	·7
May	·9	·8	·7	·3	·6	·5	1·0	·8	3·0	1·7	1·2	2·2	·7	1·1	1·2	1·1	·7	·6	·9	·4	·5	·7	·1	·3
June	2·5	2·2	4·8	2·8	1·7	3·1	5·2	2·6	11·2	2·6	3·9	14·4	2·2	3·7	3·3	·9	1·9	1·9	2·0	2·0	3·5	2·0	·1	·6
July	6·8	7·0	7·9	5·2	6·4	2·4	11·4	8·5	17·2	9·0	9·2	40·4	5·3	12·6	7·9	4·6	6·3	6·2	8·1	4·9	9·5	2·7	1·5	3·0
August	6·8	5·6	5·3	4·4	3·9	2·9	8·6	5·9	15·6	7·6	8·5	29·6	4·7	8·9	7·1	2·4	4·6	5·5	4·9	3·5	4·9	3·1	·6	1·9
September ...	2·9	4·0	2·9	1·6	2·6	1·8	3·8	3·5	5·7	2·8	4·3	11·5	2·9	2·4	3·9	2·4	2·0	2·3	2·9	1·4	2·0	·9	·5	·9
October ..	·3	·0	·1	·0	·5	·0	·2	·2	·3	·0	·0	·6	·2	·1	·1	·6	·2	·1	·2	·1	·2	·0	·0	·0
November ...	·0	·0	·0	·0	·0	·0	0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
December ...	·7	·5	·4	6	·8	·6	1	·3	·4	4	·7	1·1	·5	·4	·4	·4	·4	·5	·7	·4	·4	·4	·3	·3
Mean Year. .	24	23	26	17	19	13	36	27	63	28	33	115	20	37	31	15	20	22	28	17	29	12	6	11
Min. Year ...	8	12	11	7	8	8	18	16	51	16	25	67	14	23	23	9	11	12	21	13	21	5	4	3
Max. Year ...	3	30	38	31	28	20	51	46	74	45	43	191	28	47	43	20	35	32	31	20	32	25	16	14

V.—[illegible] .ontinued.

Place	Montgomery	Muzaffargarh	D. Ismail Khan	D. Ghazi Khan	Bannu	Peshawar	Kohat	Hazara
Latitude			32° 0′	30° 6′	32° 58′	34° 0′	33° 17′	34° 10′
Longitude			71° 5′	70° 42′	70° 35′	71° 29′	71° 30′	73° 0′
Height			600			1056		
No. of Years	6	6	6	6	6	6	6	6
January	·1	·5	·8	·4	·5	·8	·8	2·9
February	·2	·3	·6	·1	1·1	1·2	2·1	4·3
March	·7	·6	1·2	1·0	1·6	1·4	2·2	6·6
April	·2	·4	·5	·4	1·1	1·3	1·9	2·8
May	·1	·1	·4	·0	·5	·5	1·1	2·9
June	·6	2·0	1·2	·6	1·0	·2	1·1	4·3
July	3·7	1·0	1·4	1·3	1·4	1·1	4·0	9·3
August	2·5	1·7	·7	1·0	2·1	2·1	2·3	6·9
September	1·9	·7	·5	·5	·3	1·5	1·6	3·6
October	·0	·0	·0	·0	·0	·3	·5	·9
November	·0	·0	·0	·0	·0	·0	·0	·1
December	·3	·5	·3	·3	·2	·1	·5	1·7
Mean Year	10	8	7	6	10	12	19	48
Min. Year	4	3	5	2	6	8	9	32
Max. Year	27	14	11	12	16	17	49	69

VI.—Madras.—Mean Monthly Rainfall before 1861.

Place	C. Comorin	Alleppy	[illegible]	Trevandrum	Palamcottah	Quilon	Cochin	[illegible]	Trichinopoly	Coimbatore	Dodabetta	[illegible]	Cannanore
Latitude	8° 4′	9° 30′		8° 30′	8° 43′	8° 53′	9° [illegible]	9° 17′	10° 50′	11° 0′	11° 25′	11° 40′	11° 52′
Longitude	77° 34′	76° 20′	…	77° 0′	77° 48′	76° 39′	76° 0′	78° 16′	77° 11′	77° 0′	77° 25′	75° 46′	75° 30′
Height	50	5			0	40	6	…	250	.	8640		15
No. of Years	5							5	7	3	8	14	17
Actual Years	1843–47							[illegible]	1843–48	1844–46	1847–48	1840–53	1843–48 1850–59
January	·2	2·4	·9	1·8	2·7	·9	·7	1·3	2·0	·7	·1	·0	·3
February	·0	1·2	·4	·2	1·2	·5	·0	·3	·2	·2	7·4	·1	·2
March	·9	4·3	1·1	1·1	3·6	1·9	2·0	1·6	·5	·8	3·6	·3	·4
April	·8	3·4	·4	3·5	10·7	3·5	3·2	2·2	1·4	·5	19·8	1·2	3·7
May	4·6	26·1	4·0	10·2	5·8	16·5	17·8	3·6	8·7	1·2	4·9	6·9	13·6
June	4·6	26·0	2·0	13·1	·7	17·9	30·7	4·3	1·3	1·5	4·6	31·1	36·0
July	·7	18·2	5·1	6·4	11·0	19·9	17·3	3·9	2·6	1·6	7·4	38·9	38·5
August	·4	9·2	·1	3·3	·1	20·1	15·0	1·1	1·9	·2	9·3	22·5	16·1
September	·4	7·0	·9	3·5	1·6	3·3	4·8	1·8	5·3	5·3	7·5	12·3	5·8
October	8·8	12·8	7·3	12·3	9·6	9·9	9·7	8·2	5·6	4·5	13·5	6·2	9·0
November	3·9	6·0	3·6	4·1	14·7	3·9	5·3	5·9	2·6	·5	11·9	2·5	2·6
December	2·8	2·9	2·9	4·5	4·8	1·7	1·5	3·0	4·1	3·0	12·1	1·2	1·7
Year	28	119	29	65	57	86	105	37	36	21	102	123	128

Latitude	12° 24′	13° 0′	13° 2′	13° 4′	13° 6′	14° 20′	14° 31′	15° 50′	16° 10′	16° 20′	17° 0′	17° 4′	17° 41′	19° 20′	11° 25′	11° 20′	11° 30′	12° 25′	12° 58′	12° 18′	...	13° 57′
Longitude	75° 48′	80° 15′	80° 10′	80° 14′	80° 21′	80° 0′	75° 51′	78° 5′	81° 13′	80° 30′	77° 10′	82° 14′	83° 21′	84° 50′	76° 45′	77° 0′	77° 0′	75° 40′	77° 39′	76° 40′	...	75° 30′
Height	4500	60	...	...	..	50	1831	800	...	100	1500	50	...	112	...	...	...	...	3030	...	...	...
No. of Years	3	7	7	31	37	2	3	3	7	5	19	3	3	2	7	1	11	5	30	30	30	30
Actual Years.	1838–40	1843–48	1847–48 1855–59	1811–41	1804–7 1813 45	1859–60	1857–59	1857–59	1843–48	1855–59	1841–59	1857–59	1847–48	1859 60								
January	·0	1·9	·1	1·2	1·1	·6	·6	·0	·2	·0	·2	·0	·2	·0	·2	2·0	1·5	·0	·3	·1	·1	·0
February	4	·3	·1	·1	1·2	·0	·6	·0	1·6	·0	·0	·0	·3	·1	·4	3·0	1·2	·0	·2	·2	·3	·2
March	1·5	·4	·0	·7	·4	·0	·1	·0	·3	·9	1·0	1·0	2	·0	1·0	6·0	2·2	·3	·6	·5	·6	·2
April	2·6	·3	1·2	·4	·5	1·5	·6	2·7	·2	2·5	1·0	2·5	2·3	·0	4·2	10·0	5·7	2·1	1·4	2·7	1·5	1·9
May	7·4	2·5	1·9	1·1	2·4	·9	·4	8·1	1·9	4·6	2·1	1·0	2·0	·0	5·8	2·0	7·9	6·4	5·0	5·5	4·7	3·2
June	30·4	·9	2·0	1·5	1·6	1·5	·8	14·4	4·5	4·1	1·8	6·1	3·6	·0	6·5	2·0	6·2	15·3	3·2	1·6	2·6	4·0
July	55·9	2·3	4·3	3·7	3·1	8·7	·3	17·2	4·6	5·9	1·7	10·0	3·2	10·3	4·1	4·0	10·3	34·0	3·9	2·2	3·4	6·1
August	27·0	2·4	3·8	4·8	4·6	5·1	·9	8·4	4·9	7·7	3·0	6·5	4·0	7·1	5·1	2·0	12·5	23·4	6·0	3·2	5·0	3·7
September	11·9	3·7	4·5	4·3	5·1	3·4	·1	8·0	5·9	5·7	3·8	5·6	6·4	7·1	6·6	2·0	8·5	7·5	6·3	4·1	5·3	2·7
October	4·6	8·4	11·3	11·1	10·0	4·3	·5	8·7	6·7	7·0	5·6	9·7	14·2	2·4	8·3	10·0	13·0	9·1	6·0	6·7	7·2	4·9
November	1·4	10·2	14·3	14·1	12·4	32·1	·1	4·4	1·1	2·1	1·1	·0	4·0	·1	4·1	2·0	8·0	1·5	1·6	1·8	1·1	1·1
December	·2	8·4	6·1	4·6	5·1	2·0	·0	·0	1·9	1·0	·4	·5	·5	5·9	1·4	5·0	4·7	·1	·8	·5	·3	·3
Mean Year	143	42	50	48	46	60	4	72	34	42	22	43	41	33	48	50	82	100	35	29	32	28
Min. Year	...				18										...	...	61	80	16	12	14	15
Max. Year	...				89										...	...	101	119	55	53	57	43

MINOR PROVINCES.—Mean Monthly Rainfall of 24 places in the Central Provinces.

Place ..	Sagar	Damoh.	Marwara.	Khandwa.	Hoshangabad	Narsinghpur.	Jabalpur.	Betul.	Pachmarhi.	Laknadon.	Mandla.	Burhanpur	Maltai.	Chindwara.	Seoni	Balaghat B.	Bhandara.	Nagpur.	Wardha.	Chanda.	Sironcha.	Bilaspur.	Raipur.	Sambalpur.
Latitude	23° 49'	23° 48'	.	22° 6'	22° 45'	22° 57'	23° 9'	21° 52'	22° 27'	.	22° 37'	21° 18'	21° 47'	22° 3'	22° 6'	.	21° 11'	21° 10'		19° 56'	18° 49'	22° 6'	21° 20'	21° 31'
Longitude	78° 48'	79° 25'		79° 33'	77° 46'	75° 10'	79° 59'	77° 50'	78° 27'		80° 36'	76° 15'	78° 18'	78° 57'	79° 33'	..	79° 45'	79° 11'	.	79° 19'	79° 56'	82° 22'	81° 42'	84° 1'
Height ...	1944		.	1037	1009		1351	...	3540	.	...	.	...	^	2030	...	...	1025	...	632	.	...	980	451
No. of Years	1	1	1	1	14	1	18	6	1	1	4	1	[illegible]	[illegible]	[illegible]	[illegible]	1	10	1	6	6	1	3	5
Actual Years	1871	1871	1871	1871	1855–68	1871	1850–67	1862–68	1871	1871	1865–68	1871	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1871	1863–68	1863–68	1871	1866–68	1864–68
January ...	1·0	·6	4·0	3·2	·7	1·0	·1	2·0	·5	·4	·1	·	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	·1	·0	·3	·0	·2	·6
February ..	·4	·1	·3	·0	·6	2·4	·1	·5	·2	·4	·8	·0	1·2	·3	·4	·7	·4	·2	·1	·0	·9	·0	·1	2·4
March ..	·0	·0	·0	·0	·3	·0	·0	·0	·0	·0	·9	·0	·0	1·0	1·2	·0	·0	·6	·0	·0	·9	·0	·9	·9
April	·1	·0	·0	·0	·1	·0	·3	·0	·0	·0	·6	·0	0	·7	·7	·0	·2	·8	·1	·5	·4	·2	1·6	1·3
May	1·1	1·1	1·3	·6	·4	1·9	·0	·4	1·0	2·8	·0	·5	·8	·8	1·0	·9	·8	·9	1·3	1·0	1·8	1·1	8	1·5
June	14·7	11·9	14·3	2·4	6·1	20·2	10·2	11·4	24·5	11·4	9·7	2·4	10·4	7·6	7·2	29·5	17·6	8·2	4·8	12·1	5·0	10·9	7·6	9·3
July	24·1	13·9	11·0	3·4	16·5	20·7	25·4	12·2	38·4	17·6	19·9	2·6	7·0	11·8	15·8	21·2	19·2	11·5	13·9	11·2	12·3	14·7	10·3	11·1
August	15·3	13·6	6·1	3·9	16·4	9·8	14·5	3·7	13·1	10·5	12·3	·6	3·1	10·9	12·0	12·9	3·8	9·6	1·2	11·3	10·6	6·1	9·8	13·9
September ..	11·4	14·8	13·1	7·6	6·1	13·2	4·7	15·6	27·7	8·2	7·2	5·1	13·2	7·4	6·3	10·4	10·4	6·6	7·4	7·8	10·2	8·6	6·8	6·8
October	·0	·0	·0	·0	·7	·0	·2	·0	·0	·0	·2	·0	·0	·9	·7	·0	·0	1·0	·0	1·0	·7	·0	2·2	·5
November ...	·3	·0	·0	·2	·3	·0	·0	1·2	·0	·0	·0	1·0	·0	·5	·3	·0	·0	·0	·0	·2	·1	·0	·4	·2
December ..	·5	·0	0	·1	·1	·0	·0	·0	·8	·0	·0	·0	·0	·5	·6	·0	·0	·1	·0	·0	·0	·0	·0	·0
Year	69	56	50	21	48	70	56	47	106	51	52	13	37	44	47	67	52	41	29	43	41	42	41	49
Previous Mean	55	48	51	40	.	48			46	46		27	28			64	47		38			42		

Place	Sicandarabad.	Sicandarabad.	Pusad.*	Wun.*	Basim Town.*	Mehkar*	Janaphal.	Darwa.*
Latitude	17° 27′	17° 27′	19° 55′	20° 3′	20° 7′	20° 1[illegible]′	20° 18′	20° 19′
Longitude	78° 28′	78° 28′	77° 39′	79° 1′	77° 12′	76° 38′	76° 39′	77° 50′
Height	1830	1830	...	863	...	...	2050	1255
No. of Years	10	20	3	11	10	6	3	8
Actual Years	1861 70	1841 60	1868–70	1860 70	1861 70	1865–70	1865–67	1863–70
January	·3	·2	·5	·3	·4	1·0	2·6	·2
February	·2	·4	·0	·3	·2	·2	·0	·2
March	1·1	1·1	·4	·9	·3	1·1	1·2	1·0
April	·5	·9	·0	·0	·0	·1	·5	·0
May	1·3	2·2	·1	·4	·0	·2	·0	·1
June	3·8	3·4	3·3	7·1	6·0	4·8	3·9	6·8
July	5·6	4 9	10·9	10·9	10·3	8·9	9·3	6·9
August	5 8	5·1	5·3	10·4	6·0	6·7	11·0	7·2
September	5·5	4·6	4·8	7·5	4 6	2·5	·9	7·3
October	2 6	3 9	2·3	1·2	·8	2 1	1·8	·9
November	·7	·7	·3	·2	·0	·2	·1	·2
December	·1	·1	·6	·2	·4	·0	·1	·3
Year	27	27	29	39	29	28	31	31
Year	21	13	19	26	18	21	18	7
Year	33	43	37	64	47	34	37	51

Place	Chiki.*	Yotmal.	Karinja.*	Buldana.	Tullagaon.*	Akola.	Murtizapur.*	Malkapur.*
Latitude	20° 21′	20° 24′	20° 29′	20° 34′	20° 41′	20° 42′	20° 44′	20° 54′
Longitude	76° 19′	78° 12′	77° 3′	76° 24′	78° 9′	77° 4′	77° 26′	76° 17′
Height	..	1583	...	2174	1051	930	...	...
No. of Years	6	6	4	3	11	8	7	3
Actual Years	1865–70	1865–70	1860–63	1868–70	1860–70	1863–70	1864–70	1868–70
January	·9	·6	·0	2·0	·7	·4	·4	·1
February	·0	·0	·0	·0	·0	·2	·1	·0
March	2·3	·6	1·0	·3	·3	·9	1·1	·1
April	·5	·0	·1	·0	·0	·0	·0	·0
May	·1	2·4	·6	·4	·2	·2	·0	·0
June	6·0	9 5	3·9	8·2	4·3	3·8	5·4	7·1
July	6 0	15·6	9·4	8·5	7 4	6·6	5·8	5·6
August	6·6	12·1	9·0	6 2	9·4	6·5	10·3	5·7
September	3 2	4·9	4 5	4·2	4·5	2·3	2 2	5·7
October	2 2	·9	1·4	4·3	·8	1·2	1·4	2·8
November	·1	·2	·8	·6	·0	·3	·1	·1
December	·6	·2	·0	·2	1	·6	·1	·3
Year	28	4[illegible]	31	35	28	23	27	27
Year	10	34	23	25	12	10	16	16
Year	36	72	43	39	49	40	38	37

Place	Umraoti Town.*	Umraoti Station.	Daryapur.*	Ellichpur.*	Morsi.*	Melghat.*	Punad.*	Basim Station.
Latitude	20° 55′	20° 55′	20° 56′	21° 16′	21° 20′	21° 42′	...	20° 7′
Longitude	77° 49′	77° 49′	77° 23′	77° 35′	78° 5′	77° 13′	...	77° 12′
Height	...	...	...	...	...	...	...	...
No. of Years	7	11	3	11	10	4	5	2
Actual Years	1863–69	1860 70	1868–70	1860–70	1860–69	1866–69	1863-67	1869–70
January	·7	·7	1·0	·6	·5	·5	·0	·6
February	1	·1	·0	·1	·2	·0	·5	·0
March	1·9	·5	·2	·7	·6	·5	·3	·4
April	·2	·1	·2	·3	·0	·3	·1	·2
May	·7	·6	·0	·4	·6	·0	·1	·0
June	5·8	6·5	6·6	4·5	6·0	6·3	3·6	5·0
July	6·8	7·8	11·1	7·7	11·6	11·5	9·4	12 9
August	7·8	8·2	8·7	6·4	6·9	17·2	8·1	8·4
September	3·0	4·8	5·5	4 2	5·2	7·3	4·4	8·0
October	6	·9	3·2	1·3	·6	·8	1·2	4·1
November	·1	·2	·1	·4	·1	·0	·3	·3
December	·3	·2	·1	·1	·0	·3	·2	·0
Year	28	31	37	27	32	45	28	40
Year	17	18	30	18	15	31	22	35
Year	40	41	43	40	46	74	38	44

* From Native registers superintended by magistrates.

VII.—MINOR PROVINCES *continued*.—Mean Monthly Rainfall in British Burma.

	ARAKAN.				PEGU.					TENASSERIM.						
Place	Akyab.	Akyab.	Kyouk Phyu.	Sandoway.	Rangun.	Basim.	Hensada.	Prom.	Thayatmyo	Maulmain.	Tavoy.	Tavoy.	Mergui.	Mergui.	Shway-gyen.	Toungu.
Latitude ...	20° 8′	20° 8′	18° 10′	17° 10′	15° 20′	15° 30′	16° 4′	17° 13′	20° 18′	14° 8′	14° 7′	14° 7′	12° 27′	12° 27′	...	...
Longitude ...	92° 57′	92° 57′	90° 40′	91° 5′	92° 30′	90° 55′	92° 4′	92° 15′	[illegible]	[illegible]	[illegible]	[illegible] 18′	98° 42′	98° 42′	...	...
Height	21	21	..	...	...	...	...	...	[illegible]	[illegible]	[illegible]	[illegible]	200	200	...	...
No. of Years	11	1	1	1	1	1	1	1	[illegible]	[illegible]	[illegible]	[illegible]	7	1	1	1
Actual Years.	Before 1869.	1872	1872	1872	1872	1872	1872	1872	[illegible]	[illegible]	[illegible]	[illegible]	1853–59			
January	·4	·2	·0	·0	·0	·0	·0	·0	·0	·0	·3	·0	4·0	·5	·0	·0
February	·4	·0	·1	·1	·0	·0	·0	·0	·0	·0	1·8	·0	1·0	·0	·0	·0
March	·5	·0	·0	·0	·1	·0	·0	·0	·0	·0	1·2	·0	2·0	·8	2·1	·0
April	1·4	·1	·0	·0	1·0	1·1	·3	1·3	·1	·1	3·2	·6	3·0	3·3	1·7	·1
May	8·0	16·2	13·1	22·0	8·7	8·1	4·5	4·9	6·5	30·7	28·2	17·0	15·0	7·4	15·9	7·6
June	65·2	54·3	37·6	39·8	18·6	11·2	11·5	9·6	11·7	34·0	41·3	25·8	24·0	18·8	36·4	15·3
July	55·8	42·2	40·8	55·5	24·9	15·8	17·2	12·6	7·9	45·8	43·0	44·6	36·0	30·3	30·8	16·2
August	41·7	35·3	46·1	51·3	27·3	15·9	18·8	22·9	8·6	52·6	38·6	48·6	30·0	35·6	29·0	15·3
September ...	24·9	59·1	13·1	18·4	21·1	12·4	9·4	12·7	19·2	21·5	30·7	32·7	20·0	27·4	9·6	6·9
October	14·7	5·3	7·2	6·1	9·6	7·5	2·4	7·3	2·9	·8	10·2	7·9	24·0	10·5	11·3	9·2
November ...	6·3	7·0	1·8	1·0	1·2	1·7	1·4	0·5	1·2	·3	1·4	2·8	4·0	·8	1·0	1·3
December ...	·2	·2	·0	·0	·0	·0	·0	·0	·0	·0	·7	·0	2·0	·0	·0	·0

DAY MAXIMUM RAINFALLS.

LONG CONTINUOUS FALLS,

AND SPECIAL RAINFALL DATA.

N.B. There are not any Day Maximum Returns for Bengal Proper.
Day Maximum at Calcutta about 5 inches.

General data for extraordinary rainfall in Southern India, exclusive of very extraordinary cases.

	In 1 hour.	In 2 hours.	In 3 hours.	In 6 hours.	In 24 hours.
			Inches.		
For the sheltered table-lands of Balari and Kadapa	·75	·9	1·0	1·5	3·0
For the average of Plains like that of Tinnevelly, Ramnad, Trichinopoly, Eastern Coimbator, and Western Tanjor..............................	1·0	1·2	1·4	2·0	4·0
For the bases of hills sheltered from the S.W., but more exposed to the N.E. mansun; and for a table-land like Maisur not shut out from the N.E. mansun	1·25	1·5	1·7	2·5	5·0
For plains and table-lands of Haidarabad and Nagpur	1·5	1·8	2·0	3·0	6·0
For plains like Tanjor, South Arcot, Chinglepat; for the plains of Gantur, Nellor, Rajahmandry, Ganjam, and Masulipatam	2·0	2·4	2·7	4·0	8·0
For the hills of Kadapa, Nellor, Gantur, Rajahmandry, Ganjam, and Masulipatam...........	3·0	3·6	4·1	6·0	12·0
For hill summits well exposed to the S.W. mansun...	3·7	4·5	5·0	7·5	15·0

III.—Bombay Presidency.—Special Rainfall Data.

Day Maxima of Five Stations in Ten Years.

	1860.	1861.	1862.	1863.	1864.	1865.	1866.	1867.	1868.	1869.	Max
Belgaum	20 October 2·54	17 July 3·29	22 October 2·5			22 August ·07	17 June 4·85	18 July 2·28	6 June 3·01	28 June 5·51	6·07
Puna..	11 June 3·12	2 July 3·10	23 October 3·1			4 June 1·75	3 October 2·06	16 October 7·90	8 June 5·00	28 July 3·15	7·90
Bombay . ..	13 June 7·31	18 August 6·08	21 June 6·16	24 July 8·14	26 Jun 3·56	2 July 8·04	3 August 8·08	16 July 5·69	25 August 10·51	27 June 15·31	15·31
Karachi.	14 May 0·93	24 August 2·30	31 July 1·48	9 July 5·27	21 July 2·57	29 January 2·22	5 August 4·83	16 August 1·02	30 July 1·10	21 July 6·74	6·74
Deesa ...	...	24 August 8·19	27 July 2·56	13 August 3·54	25 July 4·61	9 July 3·77	11 August 2·80	24 August 2·25	22 August 5·15	21 July 5·31	8·19

Mahabaleshwar . 13·06 inches in 24 hours on 2nd Sept., 1833.
Sattara . . . .. 4·40 „ „ April.
Uttray Mullay ..... 15·10 „ „ 24 Oct., 1845.
Bombay.......... ... 9·43 „ „ 10 July, 1844.
„ .. .. 4·22 „ in 1 hour on 11 June, 1847.
„ . .. . 12·00 „ in 36 hours in July, 1859.
„ (near?). . . 14·00 „ in 24 hours on 9 Aug., 1868.
„ .... 15·31 „ „ 27 June, 1869.

IV.—NORTH-WEST PROVINCES AND OUDH.

Day Maxima in Six Years.

	1867.	1868.	1869.	1870.	1871.	1872.	Max.
	...	..	3·1 June	2·5 Sept.	3·8 Aug.	N.B.—The Reports for 1872 do not give any Day Maxima.	3·8
	..	...	...	.	2·1 June		2·1
	...	4·3 July	4·9 Sept.	5 9 July	7 7 June		7 7
	4·8 Sept.	3·7 July	3 0 July	4 2 July	3 6 June		4·8
	.	...	2 5 Sept.	6·1 June	3·5 July		6·1
	.	.	8 8 Sept.	8·5 June	3·8 June		8·8
	3 0 Aug.	0·7 June	2 8 Sept.	3·1 July	2 2 July		3·1
		..	6·1 July	3·7 Aug.	3 0 July		6·1
			.	3·9 Oct	7 8 July		7 8
	3·8 Sept	4 6 June	2 1 July	3·7 July	3·4 June		4 6
			3·9 Oct.	4 8 July	5 7 Sept		5·7
	8·0 Aug.	5·0 Aug.	7 9 Oct	.	..		8·0
	...	4 8 July	7·5 July	5 6 Sept.	6·2 Sept		7 5
	3 0 July	2 2 July	4 2 Sept.	7·4 Aug.	3 9 June		7·4
			2 9 July				2 9
	4·4 July	3·5 Jan.	9·3 July	2 5 July	2·5 Aug.		9 3
		..	..	.			..
	.	...	.	.	.		.
	.	.	.	6·4 July	.		6·4
	6·1 Sept.	...	...	...	.		6·1

Special Falls.

	9 00 inches in 48 hours on 9th and 10th July, 1869.				
	15·00	,,	36	,,	23rd and 24th July, 1869.
	3 80	,,	24	,, in	June, 1866.
(Ajmir)	5·70	,,	,,	,,	January, 1864.
	5·6	,,	,,	,,	August, 1863
	4·8	,,	,,	,,	September, 1865.
Tal	8 0	,,	,,	,,	December, 1863.
	5 2	,,	,,	,,	September, 1865.
	5·4	,,	,,	,,	July, 1866.
	8·5	,,	,,	,,	August, 1866.
	2·8	,,	,,	,,	September, 1863.
	2 5	,,	,,	,,	August, 1864.
	6·6	,,	,,	,,	July, 1865.
	3 0	,,	,,	,,	September, 1866.
	2 8	,,	,,	,,	July, 1863.
	2 1	,,	,,	,,	August, 1864.
	3 8	,,	,,	,,	August, 1865.
	3 6	,,	,,	,,	July, 1866.

V.—PANJAB.

Day Maxima in Four Years.

	1869.	1870.	1871.	1872.	Max.
Delhi	3·4 Sept.		3·4 Aug.	2·8 Aug.	5·5
Gurgaon	3·5 July		1·7 Sept.	2·7 June	3·5
Kurnal	3·2 July		4·4 July	3·4 June	4·4
Hissar	1·6 Sept.		1·8 June	4·5 Aug.	4·5
Rohtak	4·0 Sept.		2·6 Dec.	2·7 July	4·0
Sirsa	3·6 Sept.		1·5 June	2·7 Aug.	3·6
Ambala	2·7 July		5·4 July	2·9 July	5·0
Ludiana	5·5 July		1·8 Feb.	4·8 Aug.	5·5
Simla	2·8 July		5·6 July	3·5 June	5·6
Jullundur	4·6 July	[illegible]	3·0 Feb.	4·4 Aug.	4·6
Hoshyarpur	2·4 July	3·4 Sept.	7·5 June	5·1 July	7·5
Kangra	5·2 Sept.	3·9 July	5·8 July	7·2 Aug.	7·2
Amritsar	4·4 June	3·9 Sept.	1·6 July	3·3 July	4·4
Sialkot	7·3 June	5·0 July	3·4 Aug.	2·5 July	7·3
Gurdaspor	4·6 July	4·0 Aug.	3·4 Sept.	2·1 July	4·6
Lahor	3·6 Oct.	2·1 Aug.	1·4 July	2·3 June	3·6
Firozpor	7·0 July	6·0 Aug.	2·4 June	6·5 Aug.	7·0
Gujranwala	2·6 July	2·8 July	2·8 June	3·9 Aug.	3·9
Rawalpindi	1·9 Sept.	3·2 July	1·9 Aug.	1·7 Jan.	3·2
Jhelam	1·8 Aug.	2·2 Aug.	1·5 July	1·2 July	2·2
Gujrat	3·5 July	2·9 July	2·4 July	3·2 July	3·5
Shahpur	1·4 Aug.	1·4 Aug.	2·8 June	3·2 July	3·2
Multan	2·9 Mar.	0·5 Aug.	0·9 July	3·5 July	3·5
Jhang	2·1 July	1·0 June	1·0 Dec.	1·8 Sept.	2·1
Montgomery	3·7 July	1·9 Aug.	4·0 July	1·4 July	4·0
Muzaffargarh	1·6 July	2·0 Aug.	1·6 July	2·3 Aug.	2·3
D. Ismail Khan	1·5 June	2·3 June	0·8 June	1·4 July	2·3
D. Ghazi Khan	2·7 July	1·5 June	1·0 July	1·5 Mar.	2·7
Bannu	1·5 July	1·0 Aug.	3·0 June	1·4 Apr.	3·0
Peshawar	4·4 Sept.	1·8 Aug.	2·9 Feb.	4·1 Aug.	4·4
Kohat	8·0 Mar.	5·0 Apr.	1·6 Aug.	2·0 July	8·0
Hazara	2·0 June	2·9 June	2·9 July	3·7 July	3·7

VI.—MADRAS, MAISUR, AND CURG.

Day Maxima at Madras between 1822 and 1857.

inches on 4th November, 1822.
„ „ 29th October, 1825.
„ „ 9th May, 1827.
„ „ 27th November, 1827.
„ „ 31st October, 1836.
„ „ 20th November, 1836.
„ „ 27th December, 1845.

17·00 in 12 hours on 21st October, 1846.
20·58 in 24 hours on 21st October, 1846.
11·45 in „ 4th May, 1851.
7·90 in „ 4th November, 1851.
6·22 in 5 hours on 20th November, 1856.
12·21 in 12 hours on 24th October, 1857.
18·04 in 24 hours on 24th October, 1857.

Falls at Bangalor.

ches in 24 hours in Sept., 1852.
„ in 35 min. in May, 1859.
„ in 24 hours in Sept., 1859.
„ in 24 hours in Aug., 1860.
„ in 15 min. in Sept., 1860.
„ in 40 min. in May, 1861.
„ in 2¼ hours in Sept., 1861.
„ in 24 hours in Nov., 1861.

Longest Continuous at Bangalor.

10 days in July, 1859.
10 days in August, 1859.
10 days in August, 1860.
9 days in August, 1861.

Dodabetta ....4·30 inches in 24 hours in May, 1852.
Shemuga..................2·00 „ „ in April, 1859.
Shemuga..................4·00 „ „ in September, 1859.
Chittledrug ....10 days continuously in April, 1859.

VII.—MINOR PROVINCES.—Haidarabad and Barar.

	1863.	1864.	1865.	1866.	1867.	1868.	1869.	1870.	Max.
larabad ...	1·70	3·00	1·90	2·65	2·05	2·74	2·27	2·00	3·00
Station ...	...	...	...	...	...	...	...	4·30	4·30
Town ...	...	...	...	...	...	..	...	7·20	7·20
....	...	...	...	...	...	...	...	4·65	4·65
al	...	...	3·56	2·50	2·21	2 87	...	...	3·56
l............	...	...	6·30	4·30	6·40	4·35	3·60	7·40	7·40
a	...	...	...	...	...	4·05	4·53	4·91	4·91
....	9·72	2·20	2·67	4·22	1·60	4·59	3·10	4·27	9·72

Longest Continuous Falls at Sikandarabad.

7 days 2·32 inches in 1863.
6 „ 5·50 „ 1864.
0 „ 2·11 „ 1865.
7 „ 1·17 „ 1866.

5 days 1·53 inches in 1867.
7 „ 3·23 „ 1868.
8 „ 3·38 „ 1869.

Central Provinces.

Day Maxima in 1869.

Nagpur	2·04	inches in	September.	Raipur	9·70	inches in	July.
Jabalpur	4·20	,,	July.	Seoni	4·10	,,	October.

Day Maxima in 1871.

Sagar	4·90	inches in	July.	Nagpur	4·02	inches in	June.
Jabalpur	3·83	,,	August.	Chanda	4·00	,,	September.
Hoshangabad	7·50	,,	Septem	aipur	2·88	,,	September.
Pachmarhi	4·85	,,	Septem	abalpur	2·88	,,	September.
Seoni	2·50	,,	June.				

B mah.

There are not any Da e reports of this Province.

TABLES OF
HUMIDITY AND EVAPORATION.

Place......	Dodabetta.	Madras.	Belgaum.	Ballari.	Sicanderaba	Sattara.	Dapuli.	Mahableshw	Puna.	Serur.	Bombay.	Dhulia.	Rajcot.	Ahmadnagar.
Latitude	11° 25′	13° 6′	15° 52′	17° 0′	17° 28′	17° 40′	17° 48′	17° 59′	18° 30′	18° 50′	18° 53′	20° 54′	22° 18′	23° 34′
Longitude ...	77° 5′	80° 21′	72° 42′	77° 0′	78° 32′	74° 2′	73° 16′	73° 30′	74° 0′	77° 25′	72° 52′	74° 45′	70° 50′	73° 1′
Height	8′40	27	2260	1500	1800	2320	600	4500	1800	1752	64	1000	450	1900
No. of Years	1	18	2	9	10	5	2	9	5	5	12	6	4	6
Actual Years.	1847	1833–50	1858–59	1851–59	1850–59	1855–59	1858–59	1835–43	1856–60	1854–58	1847–58	1853–58	1857–60	1854–59
January	64	75	51	44	49	65	79	53	53	58	70	65	41	50
February ...	93	71	37	50	47	62	79	44	40	42	67	52	35	43
March.........	70	72	44	34	45	56	80	40	44	15	67	45	33	38
April	81	76	48	60	37	48	80	41	43	25	72	46	35	35
May	87	69	63	51	38	60	85	59	57	43	68	47	40	43
June	93	64	79	58	49	79	58	83	67	56	80	65	55	63
July...........	93	68	84	71	60	75	85	88	79	[illegible]	85	76	64	75
August.........	93	72	84	60	72	89	90	94	84	71	80	85	72	71
September ..	93	76	79	45	70	84	89	83	79	[illegible]	85	85	60	79
October	93	76	62	67	63	74	80	88	63	[illegible]	76	71	45	63
November ..	93	79	62	79	55	66	89	64	52	[illegible]	67	59	33	58
December ..	86	75	54	65	51	69	70	60	53	28	67	57	36	53
Average	87	73	63	57	53	69	80	66	59	48	74	63	46	56

I.—INDIA.—Comparative Humidities for 27 place

Place	Karachi	Deesa	Haidarabad	Thayat Myo	Calcutta	Fort William	Hazaribagh	[illegible]	[illegible]	Landaur	Peshawar
Latitude	24° 51′	25° 14′	25° 30′	20° 18′	22° 34′	28° 34′	2[illegible]	[illegible]	59′	30° 27′	34° 20′
Longitude	77° 2′	72° 5′	69° 5′	92° 46′	88° 25′	88° 85′	[illegible]	[illegible]	56′	78° 10′	71° 29′
Height	27	400	99	240	18	18	[illegible]	[illegible]	900	7000	1056
No. of Years	5	3	2	1	2	6	[illegible]	[illegible]	1	4	3
Actual Years	1856–60	1857 59	1856 57	1859	1843–44	1854–59	[illegible]	[illegible]	1859		1858–60
January	49	52	59	41	70	[illegible]	[illegible]	[illegible]	77	49	67
February	57	46	50	32	63	[illegible]	[illegible]	[illegible]	73	65	69
March	53	36	44	27	69	[illegible]	[illegible]	[illegible]	69	45	77
April	54	34	43		70	[illegible]	[illegible]	0	40	42	74
May	65	45	44	58	77	[illegible]	[illegible]	47	53	53	48
June	65	56	50	68	81	[illegible]	[illegible]	94	59	58	53
July	72	72	59	80	85	[illegible]	[illegible]	94	77	78	56
August	76	80	65	76	90	[illegible]	[illegible]	94	64	82	61
September	68	72	68	76	90	[illegible]	[illegible]	94	80	78	49
October	57	51	60	80	80	[illegible]	[illegible]	75	84	65	37
November	39	37	48	70	80	[illegible]	[illegible]	74	78	57	38
	[illegible]	[illegible]	56	66	74	[illegible]	[illegible]	84	62	54	47

—Comparative Humidities (two observations)

Place	Galle.	Colombo.	Trincomalee.	Jaffna.	Port Blair.	Madras.	Hazaribagh.	Patna.	Goalpara.	Gorakpur.	Allahabad.	Jhansi.	Lakhnau.	Agra.
Latitude	6° 2′	6° 55′	8° 35′	9° 40′	11° 41′	13° 5′	24° 0′	25° 37′	26° 11′	26° 46′	25° 26′	25° 27′	26° 50′	27° 10′
Longitude	80° 20′	79° 45′	81° 25′	80° 0′	92° 42′	80° 70′	85° 24′	85° 8′	90° 40′	83° 18′	81° 45′	78° 37′	81° 0′	78° 5′
Height	40	42	175	9	61	27	2014	179	386	340	289	940	364	551
No. of Years	4	3-4	3-4	3	6	6	7	6	5	5-6	5-6	6	6	6
Actual Year.														
January	85	81	77	74	69	62	46	54	64	58	56	48	58	54
February	83	80	75	72	70	61	36	47	57	49	46	38	48	47
March	80	81	72	72	68	59	31	32	50	45	37	32	34	39
April	82	83	71	75	68	60	29	34	63	41	33	24	30	28
May	85	83	67	80	79	55	34	40	74	50	34	26	37	32
June	88	86	65	82	81	52	60	56	85	67	50	46	53	43
July	88	84	64	81	82	54	81	72	83	81	76	71	76	69
August	89	83	68	82	81	59	81	76	82	80	76	69	76	74
September	89	86	68	84	83	62	78	74	83	78	76	67	74	70
October	87	83	74	80	80	69	56	54	74	57	54	41	52	45
November	87	83	85	82	73	72	44	46	69	50	47	34	45	40
December	86	81	81	81	70	67	42	51	67	57	53	42	53	50
Average	86	83	72	79	75	61	51	53	71	59	53	45	53	49

I.—INDIA.—Compar. .umidities (two observations) of 27 Places, between 1867 and 1873 inclusive—*continued.*

Place …	Ajmir.	Bareli.	Mirut.	Sagar.	Jaba'pur.	Hoshang bad.	Chanda.	Raipur.	Seoni.	Nagpur.		FOUR OBSERVATIONS.		
												Viagapa-tam.	Banaras.	Rurkhi.
Latitude ……	26° 27'	28° 20'	29° 55'	23° 49'	23° 9'	22° 45'	19° 56'	21° 20'	22° 6'	21° 10'	Humidities given :— The following Three Places ha	17° 42'	25° 20'	29° 52'
Longitude ..	74° 40'	79° 25'	77° 40'	78° 48'	79° 59'	77° 46'	79° 19'	81° 42'	79° 33'	79° 11'		83° 22'	83° 2'	77° 56'
Height ….	1800	570	739	1944	1351	1009	632					31	263	878
No. of Years.	6	5–6	5	3-4	5	4	3-4					4	4–5	6
Actual Years														
January …	45	61	53	37	41	44	52	41	40	41		65	66	64
February ….	40	52	46	26	35	29	43	32	34	33		65	59	62
March ……	35	43	38	16	30	22	38	30	27	27		65	44	51
April …….	30	33	28	16	24	22	39	26	21	23		67	39	37
May …	33	36	37	19	22	23	40	25	21	23		68	40	37
June ….	44	53	45	45	44	48	57	51	51	50		68	58	52
July ……	65	76	71	78	65	78	75	78	79	72		73	82	78
August ..	67	75	71	78	66	81	71	74	78	69		73	82	78
September ..	58	74	67	71	67	73	75	74	76	71		75	81	76
October …..	35	58	47	32	53	48	59	54	50	48		73	68	61
November ..	35	50	43	27	41	40	56	41	41	45		66	68	57
					[illegible]	[illegible]	54	43	43	46		64	68	64

	Bombay.					North-West Provinces.										
Place	Belgaum.	Puna.	Bombay.	Karachi.	Deesa.	Chakrata.	Ranikhet.	Dehra.	Rurkhi.	Mirat.	Bareli.	Agra.	Fattehgarh.	Lakhnau.	Allahabad.	[illegible]
Latitude ..	15° 52′	18° 30′	18° 53′	24° 51′	25° 14′	30° 40′	29° 38′	30° 20′	29° 53′	29° 41′	28° 21′	27° 0′	27° 23′	26° 50′	25 26	25 [illegible]
Longitude .	72° 42′	74° 0′	72° 52′	77° 2′	72° 5′	77° 55′	79° 29′	78° 8′	77° 54′	77° 41′	79° 27′	78° 0′	79° 48′	81° 0′	81 52	83 0
Height	2260	1800	64	27	400	7056	6143	2232	880	739	570	551	503	364	289	[illegible]
No. of Years.	10	10	10	10	10	4	2	5	5	4	5	5	5	5	4	5
Actual Years.	1860–69	1860–69	1860–69	1860–69	1860–69	1869–72	1871–72	1868–72	1868–72	1869–72	1868–72	1868–72	1868–72	1868–72	1868 72	[illegible]
January ...	59	61	68	60	69	50	51	56	57	52	63	55	52	60	55	52
February	53	58	66	58	59	55	50	56	55	46	42	48	41	49	45	[illegible]
March ...	46	50	67	61	50	48	39	48	41	38	45	39	32	35	37	[illegible]
April	48	47	70	63	47	40	39	41	29	29	36	29	31	28	35	[illegible]
May	59	52	67	66	53	43	51	36	29	37	36	27	29	35	34	[illegible]
June	80	73	78	62	64	66	71	59	49	49	56	44	50	55	57	[illegible]
July	88	80	85	74	75	88	82	79	69	70	73	67	76	75	75	[illegible]
August	88	82	86	76	83	91	84	81	69	71	72	72	74	76	75	[illegible]
September ..	84	78	84	72	67	84	77	76	65	66	74	68	71	75	76	[illegible]
October ...	74	71	78	51	63	58	48	55	46	46	56	44	44	53	55	[illegible]
November ...	62	61	76	53	58	45	43	47	41	42	51	38	39	45	46	[illegible]
December ..	63	63	61	57	47	55	50	55	51	51	58	48	47	54	52	[illegible]
Average	67	64	74	64	62											

I.—INDIA. rative Day Humidities (two observations) for various Stations of modern date—*continued.*

	NORTH-WEST PROVINCES.			PANJAB.										CENTRAL PROVINCES.	
Place	Gorack-pur.	Jhansi	Ajmir.	Amritsar.	Dera I. Khan	Dharmsala.	Lahor.	Ludhianah	Murri.	Multan.	Rawalpindi	Sialkot.	Sagar.	Jabalpur.
Latitude	26° 46′	25° 27′	26° 27′	31° 40′	32° 0′	32° 8′	31° 34′	30° 55′	33° 35′	30° 12′	33° 4′	32° 29′	23° 49′	23° 9′
Longitude ..	83° 18′	78° 37′	74° 40′	74° 50′	71° 5′	76° 15′	74° 21′	75° 54′	73° 20′	71°, 25′	73° 5′	74° 35′	78° 48′	79° 59′
Height	255	940	1800	...	600	..	1000		.	...	1700	900	1944	1351
No. of Years.	5	5	5	5	5	5	5					5	2	2
Actual Years	1868-72	1868-72	1868-72	1867-71	1867-71	1867-71	1867-71					1867-71	1870-71	1870-71
January	58	42	44	61	51	77	56	56	50	48	50	50	44	45
February	50	37	39	66	50	66	53	54	60	45	50	45	26	40
March	42	29	35	48	47	45	48	47	45	36	44	46	19	28
April	41	23	30	...	38	49	51	32	51	28	36	40	19	23
May	52	20	23	..	30	54	23	30	39	21	32	24	26	26
June	68	41	45	58	34	17	32	39	60	26	29	35	54	53
July	80	66	63	53	45	87	48	55	74	34	41	57	78	67
August	81	66	66	60	49	87	51	54	68	38	46	62	76	67
September ...	80	64	57	50	45	70	53	55	60	35	43	54	70	71
October	59	39	37	35	39	68	32	34	43	30	33	37	38	56
November ..	51	30	35	37	45	..	36	32	43	33	40	41	31	45
December ..	59	38	43	54	50	.	48	44	64	41	40	50	34	29
Average .	.	..	.	44	44	67	43	44	55	35	40	45		47

I.—INDIA.—Comparative Day Humidities (two observations) for various

	CENTRAL PROVINCES.						BERAR.			MAISUR.		BRITISH BURMA.	MADRAS.
Place	Hoshangabad.	Pachmarhi	Seoni.	Nagpur.	Chanda.	Raipur.	Umraoti.	Akola.	Buldana.	Bangalor.			
Latitude	22° 45′	22° 27′	22° 6′	21° 10′	19° 56′	21° 20′	20° 55′	20° 42′	20° 34′	12° 58′	N.B.—At this place the humidities were taken inside a house.	The humidity returns for this province for 1872, are generally untrustworthy. Humidity of Akyab is included in the returns for Bengal.	There are not any humidity returns for Madras.
Longitude ...	77° 46′	78° 27′	79° 33′	79° 19′	79° 19′	81° 42′	77° 43′	77° 4′	76° 15′	77° 39′			
Height	1009	3540	2030	1025	632	980	1162	930	2174	3030			
No. of Years.	2	2	2	2	2	2	2	2	2	2			
Actual Years.	1870–71	1870–71	1870–71	1870–71	1870–71	1870–71	1871–72	1871–72	1871–72	1866–67			
January ...	52	51	44	46	59	46	38	53	53	53			
February	29	34	34	33	45	34	28	36	45	47			
March	24	9	33	29	52	30	18	28	32	46			
April	17	10	22	26	43	28	29	31	28	52			
May	26	30	22	24	40	25	40	34	38	56			
June	57	63	63	56	58	61	54	52	56	61			
July	80	86	80	73	78	79	68	61	74	65			
August ...	83	88	76	64	68	73	67	72	72	68			
September ...	72	81	75	67	75	76	68	75	76	69			
October	51	50	55	47	59	53	40	49	49	73			
November ...	46	50	46	50	59	43	31	49	25	61			
December ...	47	45	...	44	55	42	41	49	33	66			
Average	48	...	48	47	...	49	...	...	...	...			

II.—Bengal.—Mean Humidities (four observations) of 16 Places, from 1867 to 1873 inclusive.

Place	Akyab.	False Point.	Cattak.	Sagar Island.	Chittagong.	Calcutta.	Jessor.	Dacca.	Silchar.	Hazaribagh.	Berhampur.	Gya.	Patna.	Monghir.	Darjiling.	Goalpara.
Latitude	20° 8′	20° 20′	20° 29′	21° 39′	22° 21′	22° 33′	23° 9′	23° 43′	24° 40′	24° 0′	24° 6′	24° 42′	25° 37′	25° 22′	27° 3′	26° 11′
Longitude .	92° 57′	86° 47′	85° 54′	88° 5′	91° 50′	88° 21′	89° 7′	90° 27′			17′	[illegible]	85° 8′	86° 30′	88° 18′	90° 40′
Height	21	19	80	6	90	18	20	35					179	160	6941	386
No. of Years	4-6	6	6	5-6	5 6	6	4-5	5 6					4-5	4-5	6	4-5
Actual Years																
January	76	80	68	75	71	67	69	71	80	50	67	57	68	64	76	72
February. ...	75	79	66	76	71	66	64	67	75	39	60	43	56	56	77	65
March	75	83	65	78	74	67	63	70	74	36	54	27	46	44	68	61
April	76	86	66	80	80	72	68	77	77	33	61	33	44	47	75	73
May . ..	81	85	67	81	82	75	77	84	82	40	67	47	50	54	81	79
June	87	84	75	84	86	83	82	89	88	65	80	61	63	72	90	88
July ...	89	85	82	86	88	87	87	89	88	86	86	81	81	84	91	86
August	87	85	83	87	88	88	87	89	49	85	85	81	83	84	92	86
September .	85	83	83	86	87	87	86	88	88	83	84	79	80	84	90	87
October .	85	79	78	83	83	79	82	81	86	64	76	65	70	71	77	81
November ...	81	72	69	75	78	71	76	73	82	52	71	52	62	62	72	77
December	78	75	68	75	74	69	74	71	81	51	69	57	64	62	71	73

EVAPORATION DATA.

BOMBAY, 23 YEARS.

	Observed Mean Daily Evaporation.	Mean of 23 Years' Observations. Temperature of Air.	Temperature of Evaporation.	Relative Humidity.	Rainfall.	Mean Velocity of Wind in 1867.
	Inches.				Inches.	Miles per hour
	·240	74·0	67·5	·727	·05	10·8
y	·250	75·5	68·5	·708	·01	10·4
	·283	79·2	72·4	·726	·00	12·1
	·285	82·7	76·3	·754	·05	12·7
	·322	85·3	78·6	·751	·56	10·0
	·174	83·5	79·2	·842	20·84	15·7
	·108	81·3	78·3	·877	24·26	19·4
	·135	80·3	77·3	·873	13·16	18·3
ber	·151	80·4	76·7	·858	9·64	12·7
r	·217	81·5	76·5	·810	2·20	9·8
ber	·266	79·2	71·9	·715	·47	11·4
ber	·270	76·1	69·1	·712	·05	9·0
Mean or	80·8	79·9	74·4	·780	71·30	12·7

AKOLA, 1870.

	Mean Daily Evaporation.	Total Monthly Evaporation.	Maximum Day Evaporation.	Humidities. Mean.	Humidities. Max.
	Inches.	Inches.	Inches.		
y	·258	7·75	·35	38	67
ry	·359	10·77	·45	30	43
	·472	14·16	·65	36	68
	·741	22·24	·90	27	53
	·945	28·34	1·11	32	64
	·446	14·39	·96	42	62
	·130	3·90	·36	60	77
t	·297	8·92	·42	56	75
ber	·260	7·81	·46	70	94
r	·430	12·91	·95	62	95
ber	·338	10·16	·47	58	97
ber	·352	10·56	·48	51	71
Total or Max	·417	151·9	1·11	47	97

VARIOUS EVAPORATION DATA.

	Mean daily—	·50	inches.	
hi	,,	·50	,,	or 60 inches in 4 months.
lor	,,	·22	,,	in November, vessel on ground.
lor	,,	·17	,,	,, vessel buried.
(Bombay)	,,	·125	,,	or 30 inches in 8 months (Conybeare).
generally	,,	·25	,,	according to Col. Cotton, through the year
	,,	·30	,,	according to Observatory Data.

Evaporation Data for Pondicherry, by M. Lamairesse.

	Mean Evaporation, in Inches.			Mean Temperatures Corresponding.	Maximum Evaporation, in Inches.			Temperatures Corresponding.	Minimum Evaporation, in Inches.			Temperatures Corresponding.
	Day.	Night.	Both.		Day.	Night.	Both.		Day.	Night.	Both.	
1864.												
July	·255	·202	·458	88·0	·551	·394	·906	86·5	·118	·0 once	·157	86·0
August . .	·151	·144	·294	84·4	·236	·197	·433	84·0	·039	·0 twice	·039	81·9
September	·172	·142	·314	85·1	·276	·236	·433	85·3	·078	·0 once	·157	85·6
October	·135	·106	·240	83·3	·197	·197	·394	85·3	·039	·0 twice	·039	78·6 to 79·5
November	·137	·068	·205	80·2	·197	·157	·354	83·5	·0 once	·0 once	·039	74·3
December	·132	·071	·203	78·3	·197	·0	·276	76·5 to 79·7	·039	·0 twice	·039	77·9
1865.												
January	·169	·122	·291	77·7	·276	·197	·394	77·2 to 77·9	·039	·0 once	·039	76·3
February.	·189	·125	·313	81·1	·276	·197	·394	81·7 to 83·3	·118	·078	·197	79·5
March	·188	·165	·353	83·5	·197	·197	·394	83·8	·157	·118	·276	83·1
April	·192	·157	·348	85·6	·276	·197	·394	86·2	·157	·039	·236	83·5
May	·305	·144	·449	86·0	·591	·197	·787	85·8	·157	·078	·276	84·7 to 85·8
June	·385	·127	·488	88·3	·551	·236	·591	88·7 to 89·6	·157	·0 once	·315	90·0
Year Tota's	72·5	47·0	120·3		.	...	...					

oration and Corresponding Temperatures at Red-hill, Madras. By Mr. Ludlow, C.E.

	Total Evaporation in Inches.		Daily Mean in inches.		—	Rainfall in Inches
	In tank.	In open.	In tank.	In open.	Ratio.	
....	9·921	14·173	·409	·567	1·39	0·
.....	11·181	14·252	·374	·476	1·27	2·559
.....	11·772	15·079	·406	·500	1.24	5·591
....	10·079	12·008	·327	·386	1·19	7·047
....	7·205	8·465	·358	·421	1·17	1·890
.....	50·	64·	...	...	1·25	17·126
.....	...	...	·374	·469		

	Temperature of Water evaporated.		Temperature of Water in Tank.		Temperatures.	
	In tank.	In open.	At top.	At bottom.	In sun.	Of dew point
....	83·7	86·5	89·1	85·1	106·8	78·4
.....	81·2	82·8	84·2	81·1	102·0	78·8
.....	79·3	81·7	82·0	78·8	101·3	77·1
.....	79·9	81·9	83·0	80·6	97·3	77·0
....	80·4	82·0	81·7	80·2	98·4	76·6
.....	81·0	83·0	82·3	83·0	101·1	77·5

Madras Observatory Evaporation Data.

	Mean daily.		Mean daily.
January	·300	July	·413
February	·305	August	·354
March	·359	September	·334
April	·392	October	·288
May	·460	November	·247
June	·484	December	·266

Total for year, 125·8 inches.

poration at Chandarnagar, during 66 days of cold weather, from 27th September th December, 1865, gave results varying between 24·8 and 35·4 inches; the ations being made on 60 tanks, whose surfaces varied from one quarter of an acre acres, and whose depths varied from 6 to 18·3 feet.

THE EVAPORATION DATA AND THEIR CONDITIONS OF OBSERVATION.

The data for Redhill, for Pondicherry have been
reduced to English meas e given by M. Lamairesse, in
Vol. XVIII., for 1869, of les Ponts et Chaussées." With
regard to the former, it is t the observations in the open
were made at about to hose in the tank, and that the
results show that the de io of evaporation in the open
to that in the tank is e diminution of the depth of
water in the tank. Th nk went down in five months
75 inches, in spite of 8 n all 83 inches; of which the
tank evaporator accounted for only 53 inches as lost by evaporation, hence only 30 inches were used in irrigation out of 83 inches in the tank—*i.e.*, three-eighths were utilized, and five-eighths lost.

M. Lamairesse also mentions that the English engineers in the Madras presidency also allow for a loss of water in irrigation by evaporation of 3 inches daily per square yard of land irrigated.

The conditions and mode of observation adopted in the old Bombay Observatory data, the Madras, Calcutta, and the Rurkhi data are not explained.

The Akola data were observed by a military surgeon; the evaporator being a simple tin pot, about four inches in diameter, surrounded by a little cotton-wool, and covered with a wire gauge covering to protect the water from animals; the water was measured every second day in the graduated measure used for measuring rainfall.

The conditions of observation adopted by Mr. Conybeare at Vahar are not forthcoming; but as his data more nearly represent the actual amount of evaporation from large sheets of standing water than those of others, and have been confirmed by practical results, they are exceedingly valuable.

ADDITIONAL
METEOROLOGICAL TABLES.

Place ……	Galle.	Colombo.	Trincomali.	Port Blair.	Madras.	Vizagapatam.	Akyab.	False Point.	Cattak.	Sagar Island.	Calcutta.	Chittagong.	Jessor.	Dacca.
Elevation	40	42	175	61	27	31	21	18	80	6	18	90	20	35
No. of Years	3–5	4 5	3–5	4–6	6	4	6–7	7	6–7	7	7	7	6-7	5–7
January … .	29·838	29·851	29·744	29·847	29.972	29·949	29·987	30·028	29·965	30·014	30·012	29·911	29·993	29·977
February…..	29·848	29·861	29·751	29·871	29·948	29·918	29·932	29·969	29·891	29·945	29·944	29·860	29·928	29·923
March	29·843	29·852	29·730	29·843	29·902	29·866	29·890	29·905	29·814	29·881	29·864	29·798	29·851	29·834
April . .	29·797	29·807	29·655	29·786	29·808	29·785	29·830	29·804	29·704	29·779	29·763	29·729	29·750	29·757
May . .	29·800	29·802	29·608	29·734	29·723	29·676	29·756	29·710	29·618	29·683	29·669	29·648	29·668	29·677
June … . .	29·789	29·795	29·580	29·718	29·673	29·590	29·678	29·593	29·532	29·555	29·547	29·542	29·538	29·558
July …	29·810	29·821	29·598	29·726	29·706	29·580	29·680	29·587	29·530	29·545	29·545	29·540	29·539	29·554
August …	29·813	29·823	29·607	29·735	29·732	29·637	29·708	29·647	29·585	29·599	29·609	29·585	29·603	29·605
September ..	29·842	29·850	29·630	29·761	29·659	29·690	29·759	29·706	29·667	29·679	29·684	29·649	29·681	29·684
October ..	29·839	29·850	29·658	29·788	29·821	29·798	29·841	29·844	29·805	29·829	29·837	29·761	29·830	29·816
November …	29·834	29·845	29·732	29·843	29·924	29·939	29·948	29·994	29·950	29·980	29·978	29·883	29·968	29·951
December ..	29·857	29·863	29·767	29·857	29·973	29·993	29·994	30·043	29·994	30·040	30·030	29·927	30·016	30·000
Year …	29·826	29·835	29·672	29·792	29·828	29·785	29·834	29·819	29·755	29·794	29·790	29·736	29·780	29·778

Averages of Mean Monthly Pressures—*continued.*

Place ...	Silchar.	Hazaribagh.	Barhampur.	Patna.	Monghir.	Darjiling.	Goalpara.	Benares.	Jhansi.	Lakhnau.	Rurki.	Agra.	Nagpur.	Jabalpur.
Elevation ..	89	1996	65	179	160	6912	386	263	836	364	880	532	1025	1353
No. of Years	4-5	6-7	6	6	5-6	5-7	5	6	6-7	6	7	7	5	5
January . ..	29·930	27·986	29·948	29·848	29·857	23·384	29·6				··116	29·499	28·942	28·616
February	29·885	27·938	29·878	29·794	29·783	23·364	29·5				·049	29·428	28·899	28·610
March .	29·812	27·882	29·790	29·693	29·690	23·364	29·4				·978	29·342	28·813	28·525
April	29·732	27·788	29·676	29·584	29·584	23·360	29·3				·867	29·223	28·710	28·427
May	29·647	27·702	29·582	29·482	29·498	23·335	29·302	29·398	28·804	29·090	28·752	29·102	28·629	28·326
June	29·547	27·580	29·453	29·349	29·367	23·265	29·208	29·268	28·662	29·150	28·619	28·967	28·531	28·202
July	29·542	27·573	29·470	29·358	29·374	23·261	29·190	29·273	28·673	29·161	28·634	28·975	28·537	28·200
August	29·598	27·641	29·548	29·436	29·443	23·305	29·257	29·348	28·755	29·232	28·698	29·039	28·607	28·278
September ...	29·677	27·709	29·632	29·515	29·523	23·362	29·339	29·433	28·836	29·325	28·799	29·138	28·653	28·329
October	29·792	27·843	29·775	29·682	29·685	23·424	29·452	29·600	29·008	29·488	28·904	29·315	28·801	28·499
November ...	29·931	27·990	29·916	29·832	29·833	23·466	29·589	29·752	29·148	29·646	29·107	29·469	28·939	28·634
December ...	29·972	28·015	29·969	29·883	29·883	23·437	29·627	29·802	29·198	29·698	29·151	29·512	28·972	28·653
Year	29·755	27·806	29·720	29·621	29·627	23·361	29·413	29·541	28·942	29·433	28·894	29·251	28·753	28·442

Place ……	Galle.	Colombo.	Trincomali	Port Blair.	Madras.	Vizagapatar	Akyab.	False Point	Cattak.	Sagar Islan	Calcutta.	Chittagong.	Dacca.	Darjiling.	Monghyr.	Hazaribagh.
No. of Years.	4–5	4–5	4–5	6	6	4	4–6	6	6	5–6	6	6	5–6	6	6	5–6
January ……	78·2	80·0	78·7	78·9	77·0	76·8	69·9	71·3	70·9	69·3	68·8	67·5	66·8	42·5	62·7	61·6
February……	79·7	80·6	79·6	79·3	79·3	79·1	73·2	75·3	75·2	74·4	73·4	71·7	72·3	44·7	68·4	66·1
March ……	81·5	82·1	82·2	81·3	82·7	83·2	78·7	80·4	81·1	80·5	80·3	78·1	79·6	50·8	77·5	75·1
April .. ……	82·0	83·3	84·5	83·9	85·6	86·3	83·7	84·0	86·2	84·1	84·1	81·5	82·0	56·1	84·8	83·0
May ………	82·2	83·7	85·8	81·8	88·8	88·7	84·2	86·8	89·2	85·8	85·6	82·9	83·4	60·1	87·5	85·9
June … ……	80·6	81·7	85·8	81·6	87·8	87·7	82·2	86·4	86·4	85·7	84·7	82·0	83·8	63·1	87·1	82·6
July ………	79·9	81·0	85·3	80·5	86·3	84·4	80·8	84·7	83·5	83·7	83·3	81·0	83·2	63·9	84·2	78·8
August . ….	79·9	81·1	85·9	80·4	85·9	85·4	81·0	84·8	83·3	83·7	83·2	81·2	83·5	63·9	83·9	78·0
September …	79·8	80·9	83·8	79·7	84·4	84·7	81·9	85·0	83·1	82·9	83·0	81·5	83·3	62·0	83·2	77·4
October ……	80·2	81·2	81·2	80·0	81·6	82·8	81·0	83·4	81·1	80·9	81·7	80·2	81·3	57·5	80·2	74·2
November …	79·5	81·0	78·9	80·2	78·4	79·9	77·7	77·0	74·7	74·3	75·5	74·6	75·1	50·5	71·8	68·3
December …	79·0	80·9	78·8	79·4	77·4	67·1	72·1	70·9	70·1	68·6	69·1	68·5	68·6	44·6	64·0	61·8
Year ………	80·2	81·5	82·5	80·6	82·9	82·9	78·9	80·8	80·4	79·5	79·4	77·6	78·6	55·0	77·9	74·4

Average Monthly Temperatures of 32 Places in India, from the Register of 1867-1873.—*Continued.*

Place	Patna.	Gorakpur.	Banaras.	Jhansi.	Allahabad.	Lakhnau.	Bareli.	Rorkhi.	Agra.	Ajmir.	Sagar.	Jabalpur.	Hoshangabad.	Nagpur.	Raipur.	Chanda.
No. of Years.	5 6	5	5 7	6-7	4-5	6	6 7	7	6-7	6-7	3-4	4 5	4-5	4 5	4-5	3-4
January	61·0	60·1	59·3	61·6	60·5	60·0	57·4	57·					66·9	69·8	68·0	67 8
February	65 9	65·3	67·3	67 3	65·9	66·4	62 8	62·					71 4	74·7	70·6	73 9
March	77 2	75 1	76·4	77·8	77·8	76 0	73·2	70·					79·1	81·8	79 7	81·1
April	84·3	83 6	86 5	87·2	85 4	85 9	82 0	81·1	85·8	85·8	84·3	84·4	88·3	87 5	87·7	89·5
May	88 1	88·7	91·5	94·1	91·2	91 7	88 4	88·1	92·5	91·8	88·1	90 6	93·2	93·4	92·5	91·7
June	88·2	87·2	91·2	91·6	90·0	91 7	89·2	89·9	93·1	91·7	84·6	88·1	88·1	86 8	86 7	86·8
July	84 0	82·9	85 7	83·3	83 9	86·0	83 9	84 4	86·1	84·8	76·8	78·6	78·9	78·6	78·2	79·2
August	83·8	82 3	85·0	82·0	82·3	85·9	83·1	83·9	83·8	82·4	75 4	78·1	78·7	78·9	79·4	79·3
September	83 0	82·1	84·0	82·0	82·2	83·5	82·0	82·5	82·9	84·3	76 1	78 6	79·5	78·7	79 4	78 6
October	79·0	77·8	78·8	78·8	77 8	78 6	76·1	75·5	78·9	79·4	75 9	75 8	77·2	76·8	77 0	75·9
November	69 8	68 8	68·7	72·1	68·7	68 4	66·3	64·3	69·7	70·9	71·3	66 8	72·0	71·7	70 7	71 3
December	62·4	61·0	59·9	63·7	61·3	60·6	58·8	57·8	62·1	63·6	64·4	62 2	68 7	67·9	67·4	67·5

Place	Port Blair	Jessor.	Hazaribag	Barhampu	Patna.	Monghyr.	Rurkhi.	Lakhnau.	Allahabad	Agra.	Ajmir.	Mirat.	Hoshangat	Chanda.	Akola.	Nagpur.
No. of Years	4-6	4-6	5-6	4-6	4-6	4-6	5-6	5-6	1	1	1	1	1	1	1	1
January	149·8	132·0	131·0	128·0	122·9	126·9	114·4	116·8	137	134	122	124	112	142	141	130
February ..	149·3	136·9	136·1	136·4	131·6	135·3	121·5	127·2	145	137	132	130	119	151	149	136
March	156·6	144·5	143·0	141·1	143·0	145·8	133·2	139·0	155	153	145	142	127	159	159	147
April	157·4	147·0	154·8	150·5	150·4	153·4	143·6	151·4	164	165	155	153	142	166	166	155
May	146·8	148·5	158·9	152·6	154·0	155·7	149·3	156·0	163	161	157	152	144	167	170	158
June	137·4	141·2	151·5	141·6	150·2	151·8	148·0	151·6	161	164	158	157	139	161	156	150
July	138·9	142·2	144·1	142·0	143·7	147·7	138·8	141·2	143	149	141	145	118	153	143	128
August	142·8	143·7	146·3	145·8	142·9	146·3	137·1	143·0	147	142	147	143	121	139	142	137
September ...	144·0	145·0	147·4	144·4	142·4	146·3	136·5	138·5	149	142	146	135	113	139	140	129
October	148·2	144·7	141·3	142·8	140·7	142·9	136·1	135·8	155	151	151	136	124	151	155	141
November ...	148·4	138·0	136·5	136·3	134·1	136·0	127·5	129·1	148	142	137	130	116	152	150	136
December ..	149·2	132·7	129·6	127·3	125·3	128·0	117·3	120·4	138	135	127	120	111	146	...	131
Year	147·4	141·4	143·4	140·7	140·1	143·0	133·6	137·5								

Maximum, Minimum, and Mean Monthly Readings of exposed Grass Radiation Thermometer, at eight Places in India, in 1872.

Place	Akyab.			Cattak.			Chittagong.			Manghir.			Hazaribagh.			Barhampur.			Darjiling.			Rurkhi.		
	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	M.n.	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.
January ...	47	56	40	51	63	37	53	65	44	44	54	36	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	29	33	23	38	52	29
February	48	58	40	55	68	47	57	66	47	52	61	42	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	35	44	29	44	57	36
March	59	83	48	66	71	59	64	73	55	62	69	50	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	38	46	26	51	60	38
April	67	76	55	74	77	66	72	78	64	69	74	64	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	45	51	40	58	79	48
May	71	75	67	75	80	68	72	78	65	75	83	65	69	76	62	73	80	64	48	56	44	70	82	59
June	76	78	72	77	80	72	76	79	67	79	85	73	74	81	68	78	84	73	55	57	50	76	86	68
July	75	77	73	76	78	73	76	77	72	...	..	...	73	75	71	78	80	73	57	59	55	78	85	73
August . .	76	77	72	75	79	72	75	79	71	76	79	72	72	74	69	77	79	73	55	59	53	75	78	71
September.	75	79	71	74	77	72	75	77	72	76	79	72	70	73	65	76	80	71	54	57	50	70	77	62
October ...	73	75	68	69	74	53	72	76	67	64	74	57	54	69	45	63	74	50	44	49	38	51	63	38
November.	65	73	59	57	66	48	64	75	58	55	69	49	48	59	36	50	66	44	35	44	30	40	48	33

	Vizagapatam.	Akyab.	False Point.	Cattak.	Chittagong.	Calcutta.	Jessor.	Dacca.	Hazaribagh.	Berhampur.	Patna.	Manghir.	Darjiling.	Goalpara.	Banaras.	Rurkhi.
No. of Years	4	4-6	6	5-6	5-6	6	4-5	5-6	3-4	5	4-5	4-5	6	4-5	4	6
January	·608	·566	·621	·509	·494	·473	·423	·475	·297	·410	·351	·364	·206	·432	·338	·288
February......	·655	·615	·710	·560	·567	·545	·470	·527	·277	·428	·375	·391	·224	·453	·376	·331
March	·737	·743	·875	·670	·728	·691	·612	·697	·300	·502	·425	·403	·252	·501	·363	·349
April	·850	·879	1·015	·815	·868	·853	·772	·834	·406	·744	·533	·524	·334	·650	·441	·348
May	·920	·952	1·096	·906	·938	·934	·903	·949	·525	·843	·721	·671	·423	·785	·537	·444
June	·903	·961	1·076	·940	·966	1·000	·975	1·019	·747	·974	·889	·913	·525	·903	·807	·682
July	·863	·938	1·023	·953	·942	1·008	·985	1·013	·825	·991	·974	·977	·543	·926	·977	·893
August	·899	·943	1·028	·958	·952	1·010	·989	1·033	·815	·988	·972	·985	·545	·931	·961	·896
September ..	·893	·967	1·006	·949	·943	·992	·968	1·011	·797	·962	·933	·952	·506	·901	·919	·818
October	·825	·914	·919	·824	·871	·860	·875	·876	·578	·804	·741	·730	·365	·794	·624	·518
November ...	·676	·780	·678	·591	·676	·629	·609	·636	·359	·581	·473	·487	·264	·608	·449	·332
December ...	·586	·615	·584	·498	·524	·488	·454	·501	·292	·443	·369	·372	·210	·473	·358	·297
Year	·785	·823	·886	·764	·789	·790	·753	·798	·518	·722	·646	·647	·366	·696	·596	·516

Average Diurnal Movement of Wind in Miles at 16 Places in India—1867–73.

Place	Madras.	Vizagapatam.	Cuttack.	Chittagong.	Calcutta.	Jessore.	Dacca.	Silchar.	Hazaribagh.	Berhampore.	Patna.	Monghyr.	Benares.	Rurki.	Nagpur.	Jubbulpore.
No. of Years	3	2	2–3	4–5	4–6	2–3	5	2–3	5	3–4	2–3	3	5	5·8	3–5	4–5
January ...	209·7	51·8	42·3	127·2	96·4	43·9	50·0	75·6					51·8	44·2	64·0	59·5
February	181·7	68·1	60·9	133·4	104·3	51·4	64·1	96·3					70·7	55·0	80·9	55·7
March	206·2	98·2	86·3	155·1	135·7	75·6	116·0	103·2					74·2	56·1	86·0	72·9
April	261·5	124·9	125·9	184·0	201·5	118·5	150·0	92·7	164·8	98·6	144·2	99·9	70·4	62·5	117·1	84·5
May	286·0	125·7	129·0	174·5	204·7	138·8	155·4	84·4	190·4	114·7	138·7	91·7	91·5	79·2	147·7	113·2
June	292·3	120·7	122·3	192·2	199·4	150·3	211·3	89·5	206·2	144·3	90·3	97·2	96·9	84·0	167·0	144·8
July	273·4	117·5	95·9	190·5	150·0	116·1	197·9	91·3	178·1	112·5	81·2	79·0	93·6	66·6	172·2	130·2
August	229·0	74·2	73·0	155·8	129·6	97·4	148·6	91·7	148·5	85·6	84·0	76·7	76·7	49·4	126·5	110·7
September	218·4	73·3	67·4	139·4	127·7	73·4	124·7	85·8	141·4	81·4	95·9	55·7	66·9	34·7	118·2	90·4
October	170·9	64·5	52·8	92·3	86·1	51·1	58·9	69·9	102·8	44·2	54·7	30·7	51·1	21·3	85·9	56·9
		[illegible]	[illegible]	[illegible]	[illegible]	44·6	38·9	73·4	84·3	31·9	52·4	31·7	[illegible]	[illegible]	71·8	47·1

Place.	Madras.		Vizagapatam.		Akyab.		False Point.		Cattak.		Chittagong.		Calcutta.		Jessor.		Dacca.		Silchar.		Hazaribagh.		Barhampur.	
No. of Years.	6		4		5-6		6		6		6		10		6		6		4-5		5-6		6	
	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir
January	73	N43E	47	S74E	38	N26W	30	N58E	16	N80E	50	N28W	40	N38W	57	N18W	40	N39W	35	S13E	56	N61W	55	N34W
February......	59	N88E	40	S22E	50	N21W	20	S29W	16	S10W	32	N38W	23	S79W	39	N53W	35	S79W	32	S24E	82	N60W	40	N64W
March	77	S50E	51	S45W	33	N47W	56	S50W	41	S17W	28	S69W	47	S33W	37	S80W	49	S24W	23	S39E	61	N78W	42	S81W
April	72	S40E	53	S39W	35	N83W	77	S42W	54	S10W	33	S17W	74	S3W	52	S11W	53	S8E	21	S64E	38	N76W	28	S1W
May	60	S14E	57	S42W	30	S57W	75	S34W	63	S1E	40	S14W	65	S12E	61	S12E	63	S23E	5	N84E	18	S85W	40	S41E
June	41	S54W	55	S50W	56	S1E	54	S47W	57	S25W	55	S26E	58	S5E	69	S16E	78	S20E	2	N79E	25	S27W	49	S31E
July	55	S59W	76	S74W	75	S9E	68	S64W	55	S41W	68	S43E	65	S12E	75	S15E	84	S22E	2	S13W	21	S17E	60	S43E
August	47	S57W	57	S69W	57	S1E	49	S64W	43	S45W	54	S36E	55	S18E	62	S14E	74	S12E	16	S53W	20	S43W	43	S45E
September ...	39	S41W	39	S33W	38	S9E	38	S39W	25	S	41	S38E	40	S30E	63	S21E	59	S14E	7	S22W	26	S52E	41	S56E
October	65	N39W	27	S78E	14	S39E	31	N49E	29	N35E	12	N22W	13	N53W	30	N59E	10	N66E	13	S17E	40	N57W	15	N9W
November ...	58	N20E	57	N74E	35	N14W	56	N25E	42	N14W	54	N14W	58	N19W	52	N6E	38	N13W	26	S75E	57	N46W	49	N28W
December .	66	N25E	55	N77E	40	N15W	56	N32E	33	N12E	61	N23W	60	N27W	63	N9W	47	N31W	39	S53E	61	N56W	59	N26W

The Excess of the Observations in the direction of the Resultant is shown as a percentage.
The Direction of the Resultant is computed by Lambert's formula from the number of observed winds.

Wind Resultants—*Continued.*

Place	Patna.		Manghir.		Darjiling.		Goalpara.		Benaras.		Rurkhi.		Agra.		Jhansi.		Ajmir.		Nagpur.		Jabalpur.		Seoni.	
No. of Years	5–6		4–5		6		5		9		9		6–7		3–6		6–7		5		5		4–5	
	°/o	Dir.	°/o	Dir	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.
January . . .	45	N70W	52	S70W	20	S62W	35	S84E	30	N72W	15	N61							34	N87E	29	N6W	21	N87E
February . .	41	N61W	39	S72W	24	S64W	35	S87E	37	N84W	23	N21							18	N51E	17	N25W	12	N13W
March	53	N60W	29	W	25	S72W	36	N87E	40	N81W	24	N58							32	S50E	18	S72W	6	N54E
April	36	N13W	23	N55W	26	S77W	44	S81E	40	N77W	11	N76							23	N74W	19	N78W	23	N77W
May	38	N30E	40	N74E	21	S47W	48	S79E	20	N26W	8	S30W	26	N68W	33	N62W	73	S59W	44	N48W	37	N64W	30	N49W
June	35	N52E	63	N85E	21	S12W	27	S73E	8	N30W	9	S3E	22	N48W	18	N72W	67	S60W	52	N79W	63	S81W	41	S72W
July	36	N76E	57	S86E	27	S37E	31	S35E	8	S12E	33	S42E	20	N48E	33	S81W	69	S63W	70	S89W	67	S84W	67	S72W
August	19	S51E	23	S60E	24	S22E	18	S3W	7	S14W	23	S39E	14	S37E	17	W	53	S68W	59	N73W	66	S78W	44	S81W
September ...	34	S83E	38	S80E	15	S9E	26	S68E	3	S49E	4	S24W	17	N24W	38	N51W	84	S68W	41	N39W	36	N76W	33	N55W
October	9	N29W	14	S81W	18	S68W	46	S85E	32	N78W	8	S29E	27	N85W	28	N59W	48	S87W	66	N35E	10	N20E	59	N18E
November ...	25	N46W	46	N85W	14	S54W	67	S87E	34	N75W	4	S13E	21	S78W	25	N53W	11	N37W	65	N66E	24	N36E	55	N52E
December ...	41	N70W	54	S67W	21	S28W	55	S86E	21	N79W	12	N64W	20	N65W	20	N8W	8	N73E	57	N67E	22	N43E	37	N22E

The Excess of the Observations in the direction of the Resultant is shown as a percentage.
The Direction of the Resultant is computed by Lambert's formula from the number of observed winds.

Average Monthly Serenity of 15 Stations in India—1869–73.

Place	Vizagapatam.	Akyab.	Cuttak.	Sagar Island.	Chittagong.	Jessor.	Dacca.	Silchar.	Goalpara.	Barhampur.	Hazaribagh.	Manghir.	Patna.	Banaras.	Rurkhi.
No. of Years	4	3–5	5	4–5	5	4–5	5	4–5	5	5	4–5	5	4–5	3–4	5
January	8·31	8·87	8·24	8·40	9·22	8·78	8·61	8·08	6·34	8·07	7·38	8·03	7 69	7·50	7·09
February	8·78	9·29	8·84	8·51	8·88	8·69	8·20	8·21	6·80	8·75	8·28	8·57	8·20	8·05	7·26
March	8·61	8·34	7·78	6·21	7·85	7·73	7·18	7·05	6·32	7·51	7·06	7·91	7·67	7·75	6·92
April	6·91	6·77	6·51	4·96	6·19	6·86	5·92	6·13	4·86	6·57	7·08	7·90	7·48	7·57	8·04
May	5·85	4·48	6·55	4·33	5·86	5·90	5·03	5·64	3·53	5·61	7·05	7·34	7·83	7·17	8·40
June	4·43	2·15	3·77	2·61	3·48	3·64	2·64	3·94	1·58	2·53	3·28	3·67	4·78	4·02	6·59
July	3·23	1·67	2·54	2·22	2·81	3·41	1·82	3·37	1·96	1·96	1·50	2·38	2·29	1·78	3·13
August	3·77	2·22	3·83	2·88	3·00	4·05	2·33	3·01	2·19	2·07	2·90	2·66	2·36	2·12	3·77
September	4·27	2·58	4·59	3·76	4·08	4·75	3·21	3·80	2·64	2·63	3·16	3·34	3·21	3·14	5·30
October	4·83	4·96	6·27	5·61	6·27	6·51	6·15	5·80	5·29	6·10	6·66	7·26	7·16	8·07	9·34
November	6·94	7·42	8·32	7·95	8·60	9·03	8·52	7·62	6·86	8·35	8·15	8·81	8·83	8·80	9·29
December	7·84	8·00	8·73	6·04	9·04	9·22	9·07	7·98	6·94	8·52	8·16	8·57	8·58	7·96	7·94
Year	6·15	5·56	6·33	5·29	6·27	6·55	5·72	5·89	4·61	5·72	5·89	6·37	6·34	6·16	6·92

In these tables an unclouded sky is represented by 10; an overcast sky by 0.
All the above are means of four observations daily.

GENERAL REMARKS

ON THE

METEOROLOGY OF INDIA.

I. R [illegible]

THE returns given, under [illegible] eads of mean monthly rainfall, and day maxim [illegible] erything available that would be of use to the [illegible] and irrigation engineer. The maximum and minim[illegible] rainfall for each place would have been given with the average annual rainfall in all cases, as in those of the Bombay Presidency, the Punjab, Mysore, and the minor provinces, but unfortunately they were not to be had. For the Madras Presidency, no mean monthly returns subsequent to 1861 are available. For Bengal and Burmah there are no day maxima procurable. In all cases hundredths of an inch of rainfall have been rejected, as in the first place they are unnecessary to the engineer, and in the second it would be aiming at a refinement of exactitude beyond the present powers of meteorological observation generally throughout India. As regards mean monthly returns, since it appears that the cycle of rainfall, in which maximum and minimum annual falls in India occur, is about ten or eleven years, the average for this number of years may be considered as practically correct; anything beyond that may therefore be considered unnecessary, and anything less as incomplete in that respect, and serving merely as a useful approximation.

The rainfall data given for years previous to 1861 were extracted and reduced from a Parliamentary Blue Book published in 1863; those for years subsequent to 1861 were reduced from yearly returns furnished by the meteorological reporters of the various provinces of India; or rather from such of them as could be procured, including the latest supplied by the India Office in 1874. All returns made by natives under the superintendence of the civil officials, Anglo-Indian magistrates, and hence not under the control or inspection of meteorological reporters, or other qualified meteorologists, have been generally excluded from these statistics; those for Berar, reduced and examined by myself, forming the sole exception.

The position of the places mentioned, latitudes and longitudes, have, when given with elevations, been generally obtained from the yearly returns of the meteorological reporters; in other cases, that is, when the elevation is not given with them, they may be considered as mere approximations intended to guide the reader. As regards the nomenclature of the places, a serious difficulty in a country where there are no less than twelve main widely-spoken languages, it has been found impossible to adhere rigidly to one system; that of Sir William Jones, being strictly phonetic, and when once learnt, free from all doubt, is undoubtedly the best, and has hence been generally adopted; but as so many places, as, for instance, Bombay and Calcutta, have fallen into an English form, and might hardly be recognized in the Jones form of Mumbai and Kalkatta, the old established manner of spelling these and a few other names has been adhered to.

As to the grouping of the rainfall stations, it would no doubt have been far more correct meteorologically to collect them into natural groups, as shown in the table on the following page; but for many reasons this has not been considered advisable at the present stage of Indian meteorology, and hence the following territorial arrangement has been generally adhered to:—

I. India generally, irréspective of Province.
II. Bengal (under the Government of Bengal).
III. Bombay and Sindh.
IV. The North-west Provinces.
V. The Punjab.
VI. Madras (under the Governor of Madras).
VII. Minor Provinces—including the Central Provinces, Berar and H[illegible] Oudh, Mysore and Kurg, British Burm[illegible] eylon.

In some cases, however, [illegible] the arrangements of the meteorological reporters, s[illegible] data for places in the Minor Provinces will be [illegible] d up with those of the nearest large province, as [illegible] he North-west Province, Maisur with Madras, and [illegible] Ceylon with Bengal.

TABLE OF NATU[illegible] PS OF RAINFALL

No.	Group.		Position.		Rainfall	
			Latitude.	Longitude.	Mean Annual.	Maximum.
I.	Insular	Ceylon	0° to 9°	79° to 82°	80	
II.	Detached	Burmese ..	0 to 21	90 to 100	128	185
III.	The East Coast, or ...	Madras . ..	8 to 20	78 to 85	40	80
IV	The West Coast, or ...	Bombay ..	8 to 24	66 to 76	80	312
V.	The Southern, or	Bangalor . .	8 to 15	77 to 78	50	120
VI.	....	Belgaum ..	16 to 17½	74 to 77	30	51
VII.	The South Central, or	Karnūl	16 to 18¾	77 to 78½	26	34
VIII	.. ..	Puna ...	18 to 19½	73¾ to 74½	24	72
IX.	. . . .	Nassik . .	19½ to 21	73¾ to 75½	52	83
X.	The Central, or	Nagpur	21 to 23¼	78 to 80	40	70
XI	The North Central, or	Mount Abu .	23½ to 25½	69 to 75	24	65
XII.	The North Eastern, or	Cherrapunji.	20 to 27½	91¾ to 93	78	612
XIII	. ...	Dacca	23½ to 26¼	89½ to 91½	93	
XIV.	The Bengal Coast, or .	Calcutta .	22½ to 23¼	88½ to 89½	93	
XV	....	Danapur	24¾ to 26¼	85 to 85½	30	230
XVI	The Bengal Hill, or ...	Bhagalpur.	23¼ to 23¾	87 to 88¾	45	
VII	. . . .	Bardwan .	23¼ to 23¾	87 to 88	30	
VIII.	.. ..	Murshidabad	23½ to 24¼	85½ to 88½	46	124
XIX.	. ...	Banaras ...	25 to 26¾	81¾ to 83½	42	62
XX.	The Northern, or .	Delhi	26 to 30½	76¾ to 79¾	30	96
XXI.	The North Western, or	Lahor	30½ to 31	74 to 78½	17	57
XXII.	The Western, or . ..	Indus .	25 to 30	71 to 73	9	30

As to the laws of the irregularities of rainfall over the vast continent of India, and their causes, nothing has yet been positively determined. The phenomena of the mansuns, and their causes, as well as those of the existence of the large comparatively rainless regions west of the Indus, have been familiar to every one for many years; famines, due to the periodic rainfall, being either in excess or in deficiency on the whole, or at the usual period of high rainfall, the rains being too late or too early, have existed for ages, and have continually decimated the population locally, without the causes being discovered. Sometimes the summer rainfall is thrown to the east, sometimes to the west of the Bay of Bengal:—sometimes it is scanty in Lower Bengal and abundant in Northern India, and sometimes the converse. After a few years, when a uniform and trustworthy system of meteorological observation shall have been extended all over India, it is very probable that these phenomena will be better understood: at present the record of pressure, temperature, and wind, &c., of the Presidencies of Bombay and Madras are practically inaccessible, and those of Northern India being irregular and untrustworthy, the only records that are of any value for this purpose are those under the control of Mr. Blanford, for many years Meteorological Reporter to the Government of Bengal.

From these he has been enabled to discover a most important law, viz., that the position of the circle of minimum barometric pressure in Bengal in March and April does, in connection with other meteorological data, furnish means for indicating roughly the amount and the distribution of the mansun rainfall of the year, which commences in May or June. We may, therefore, hope that in a few years it will be customary to announce every spring the probable amount and distribution of the summer rainfall over India, and thus save the large and continual losses of crops now due to a want of this knowledge.

Another most important law of rainfall, discovered by Mr. Meldrum, of the Mauritius, will probably be found to admit of application to India. Mr. Meldrum, of the Mauritius, originally established the law that the years of minimum and maximum sun-spot frequency were coincident with those of cyclone frequency in the Indian Ocean, and has lately estabished the law of the coincidence of these years with those of minimum and maximum rainfall at Port Louis. Now the years of minimum sun spot frequency are—

1833 [illegible]6 1867,

and those of maximum su[illegible]uency are—

1837 1[illegible]0 1871,

denoting a cycle of betw[illegible] eleven years. The rainfall at Adelaide, Brisban[illegible]pe of Good Hope, shows a similar periodicity of [illegible] generally, but the epochs are not coincident with [illegible]n-spot frequency; these periodicities are, therefo[illegible] to be the natural consequences of the same law.

Mr. J. Norman Lockyer, Superintendent of the Department of Science in Oudh, has attempted to apply these principles to rainfall in India; he states that the rainfall at Lakhnau was 64·6 inches in 1870 and 65·0 inches in 1871, each of these amounts being more than 22 inches above the fall of the preceding year 1869, or the two following years, 1872 and 1873, in which the falls were 41 and 34 inches; he also points out that the Madras rainfall records support the same law; they are thus :—

		Inches.	Totals.
Minimum	1843	41·	125·
	1844	45·	
	1845	39·	
Maximum	1847	81·	175·
	1848	40·	
	1849	54·	

and show an interval indicative of a periodicity coincident with that of the sun-spots.

While, therefore, it will probably be long before meteorological science and spectrum analysis together combine to discover the nature of the connection shown by these facts, in the meantime the knowledge of the periodicity of the rainfall cycle may, like that of Mr. Blanford's theory previously mentioned, become an invaluable blessing to India.

At present, neither of these theories can be considered as established, indeed the periodicity of a cycle of sun-spot frequency is not yet fully proved. All that is yet established, as proving the connection between the solar-spots and the meteorological conditions of the earth, is, that the years of sun-spot frequency generally correspond to those of maximum solar radiation temperature, of the black-bulb thermometer in vacuo; while of the fact that variation of rainfall is caused by that of temperature there is no doubt. A widely extended series of meteorological observations, in all parts of the world, will be required before this connection can be made to yield useful results.

II. EVAPORATION AND HUMIDITY.

Next to the amount and distribution of rainfall, evaporation is among meteorological data the most important to the hydraulic engineer. It is not sufficient for him to know how much rainfall may be expected at any time and in any length of time, he wishes to know how much of this has to be provided against, or how much of it he can utilise, after all losses by evaporation and absorption are allowed for. These losses, then, require to be determined, not with any theoretical degree of exactitude, but with a practical degree of accuracy that will be sufficient security against gross error or gross waste. The large number of bridges in India that have been swept away for want of sufficient waterway, and the large amount of water valuable for irrigation that has annually been allowed to evaporate in shallow tanks, are painful examples of semi-barbarous engineering management, and ignorance of physical and meteorological conditions.

The evaporation data given in the tables are exceedingly few in number, and have mostly been conducted on false principles; they do not by any means truly represent local evaporation as regards absolute amount, but are relatively useful, yielding comparative results, which, in combination with a few absolute data, and a knowledge of comparative local meteorological conditions, may be made to yield roughly approximate absolute da number of places. It is with this object that all mparative humidity data for India have been give npanying tables.

For example:—We that we require a rough approximation to the e Akola, a place for which comparative humiditie The most trustworthy data of absolute evapo ting evaporation from a large standing sheet of se of Mr. Conybeare, at Vahar, near Bombay; they give twenty inches in eight months of hot weather, or about forty inches in a year. Now, the Bombay Observatory data give mean daily evaporation data, which are among themselves and under their own conditions relatively correct, although their sum total, eighty inches, is not true in representing absolute evaporation from a sheet of standing water. We can therefore tabulate proportional mean daily evaporation for Bombay that will be absolutely correct, thus—

BOMBAY.	Comparative Evaporation Mean daily.	Absolute Evaporation Mean daily.	Relative Humidity corresponding.
January	·240	·120	73
February	·250	·125	71
March	·283	·142	73
April	·285	·143	75
May	·322	·161	75
June	·174	·087	84
July	·108	·054	88
August	·135	·068	87
September	·151	·076	86
October	·217	·109	81
November	·266	·133	72
December	·270	·135	71
Mean for Year	·225	·112	[illegible]

d assuming that the observed humidities taken at Akola d at Bombay are daily means, *i.e.* of two observations in e twenty-four hours, at 10 A.M. and 4 P.M., they admit of mparison, and we can then tabulate the true evaporation for kola, thus—

AKOLA.	Comparative Evaporation.	Humidity.	True Evaporation.	
nuary	·258	38	·129	Akola Deduced total evaporation for the year, 75·7 inches.
bruary	·359	30	·179	
arch	·472	36	·236	
pril	·741	27	·370	
ay	945	32	·473	
ne	·446	42	·223	
ly	130	60	065	
ugust	297	56	·149	
ptember	·260	70	130	
ctober	·430	62	215	
ovember	·338	58	·169	
ecember	·352	51	·176	
ean for Year	·419	47	·210	

his assumes that the evaporation at Bombay and at Akola ould be the same for the same relative humidity, viz., ·130 r 70, and the rest are therefore tabulated in proportion to the mparative daily evaporation data for each month, getting a al annual evaporation of 75·7 inches. This result, though nfessedly an approximation, is sufficiently true to be useful the hydraulic engineer, and is infinitely better than the old actice of basing comparisons of evaporation upon corre-ponding mean temperatures, or the still worse method of uming that evaporation all over India is about the same. In is way also we adopt a means of utilising the various evapo-tion data, taken under such different conditions, that have en generally hitherto thrown aside as useless.

In the future, we shall probably have a widely extended ies of evaporation observations taken all over India, under e orders of Mr. Blanford, now appointed to the new post of teorological Reporter for India. If these are conducted in perfectly uniform manner, whether the evaporators are ple tinpots, double boxes, or masonry cisterns, we shall possess most useful data for purposes of comparison, if the relative humidities and the average wind-movements be simultaneously observed ;—and from these, and with the aid of a few carefully conducted series of observations giving absolutely true evaporation, as from a sheet of standing water, we shall be able to tabulate absolute evaporation from any place in India with sufficient accuracy to serve the ordinary purposes of the engineer.

Before anything mor [illegible] can be expected, a large series of carefully conc [illegible] iments must be made in order to ascertain more [illegible] is known at present, both the relation between the [illegible] vaporation and the depth of the evaporating vessel ([illegible] und that between it and the velocity of the wind, i [illegible] s of relative humidity ;— we shall then have res [illegible] enlighten us considerably as to the conditions ur [illegible] may lose as much as half *the water we store for irrigation in India.*

The tables for humidity are intended to aid the engineer in determining evaporation data in the fore-mentioned manner; they may also be useful to the agriculturalist who requires certain hygrometrical conditions to suit various crops in different localities.

It is unfortunate that in many meteorological stations only two observations of humidity, viz., at 10 A.M. and 4 P.M., have been taken daily ; their mean represents, therefore, only the mean of the day, exclusive of the night, and is not a true daily mean for the twenty-four hours. Such means are therefore only comparative means ; the true mean is that of observations taken at equal intervals through the twenty-four hours. Those of observations taken six hours apart yield a mean differing only two per cent. from the mean deduced from hourly observations ; those of observations taken at eight hours' intervals are far less correct. There are no means of deducing a true daily mean from the two observation humidities ; these, therefore, only admit of comparison among themselves. In some cases the relative humidities are recorded as percentages of saturation, in others as decimal fractions; it has been thought best to leave them in the form in which they were recorded, as this presents no difficulty.

A table showing the average monthly values of the tension of aqueous vapour for sixteen stations in India is given among the additional meteorological tables, which are placed apart from those that are more useful to the engineer, viz., those of rainfall, evaporation, and humidity.

The hygrometrical data are simply inferential results derived from observations with dry and wet bulb thermometers, no direct determinations of the dew point by Daniell's or Regnault's hygrometer having been practised. The calculations have been made by Guyot's tables, which are computed by August's formula with Regnault's constants. In Berar, Apjohn's formula was used, and the results were hence less accurate.

In explanation of the various hygrometrical conditions that are thus reduced to figures and statistics, we may, for the sake of those that wish to add their observations to the common stock in a useful form, offer a few remarks.

The wet and dry bulb thermometers used for observation are suspended in the open air, in a thermometer shed, screened from the wind, but exposed freely to the air, the object being to ascertain the ordinary humidity in still, unconfined air. The dry bulb thermometer shows the actual temperature of the air; the wet bulb being cooled by evaporation falls in temperature, and the difference of the readings of the dry and wet bulb increases with the rate of evaporation, and this again increases with the dryness of the air, although not in the same ratio. The wet bulb is never cooled to the temperature of the dew point, but both that temperature and the weight of vapour in the atmosphere, and the relative humidity, are obtained by calculation. The readings recorded are simply those of the wet bulb and the difference between those of the wet and dry bulbs, except at hill stations in India, where a barometric reading is necessary in order to apply a correction. From these readings, taken at six hour intervals, and with the aid of Guyot's tables, useful mean humidities may be obtained.

The four most important hygrometrical elements are :—

I. The temperature of the dew point.

II. The actual amo [illegible] mixed with a certain mass of air in the [illegible] ur.

III. The amount of [illegible] ssary to saturate a certain mass of air.

IV. The relative d [illegible] dity of the air.

The temperature [illegible] int is that degree to which the temperature mus [illegible] n order to effect complete saturation of the air [illegible] temperature of the air be higher than that of the [illegible] he air is not saturated, and if after complete saturation the temperature of the air declines rain must fall. The amount of water necessary to effect saturation varies with the temperature : at 32° air is saturated by a little more than two grains per cubic foot ; at 42° by 3 ; at 49° by 4 ; at 56° by 5 ; at 61° by 6 ; at 66° by 7 ; at 70° by 8 ; at 100° by 20 grains nearly. The difference between the actual amount of water in the air, and the amount that it could hold at that temperature is the amount short of saturation ; and the ratio between the same quantities is the relative humidity. For example :—At the temperature of 32° if there be one grain of water in a cubic foot of air the relative humidity is 50 ; at 100° there must be ten grains present, to give the same relative humidity of 50.

The formulæ used for obtaining these data from the readings of the wet bulb thermometer and the difference of the wet and dry bulb, are those of Dr. Apjohn and of August—the latter are more recent and more accurate ; but to make use of them it also is necessary to have tables of elastic force of vapour corresponding to various temperatures. August's formulæ, as given in Guyot's Tables, Smithsonian Collection, 1862, are for temperatures above freezing (1) and below freezing (2) respectively,

$$(1)\quad F = f - \frac{\cdot 48\,(t - t')}{1130 - t'} \times b$$

$$(2)\quad F = f - \frac{\cdot 48\,(t - t')}{1240\cdot 2\,t' - t} \times b.$$

Where F is the elastic force of vapour at the dew point ;

f is that of saturated vapour at the temperature t' ;

t is the observed temperature of the dry bulb ;

t' is that of the wet bulb ;

and b is the mean barometric pressure which is assumed $= 29\cdot 7$ for the plains of India generally by Mr. Blanford.

Having thus obtained F, the corresponding temperature at the dew point can be got from a table (Drew's Meteorology or Guyot's tables) based on experiments on vapour elasticities. To calculate the humidity, obtain from a similar table the elastic force of saturated vapour (F′) due to the temperature (t), then the humidity =

$$\frac{100\ F}{F'}$$

If, however, the humidity alone be required, it can be obtained direct from Guyot's humidity tables, as before mentioned, without any calculation.

From Indian hygrometrical data, it appears that the air is least moist upon the average of the whole year at about two P.M., but this varies at different seasons ; the greatest moisture in the day is at about six A.M., and there is a mean state about nine or ten o'clock, both A.M. and P.M. The extremes of humidity are generally the reverse of those of temperature as regards time, except in June and July, when the moisture is greatest about midnight ; in August and September the increase of moisture after midnight is very small. The contrasts between the humidities of Madras and Bombay show the effects of the north-east and south-west mansuns. The variations of humidity from year to year at the same place seem not to follow any law, and the humidities for various places seem not to be affected by latitude or longitude. The effect of elevation is everywhere clearly shown by the almost proportional lower reading of the dew point, less water being present in the air, a nearer approach to saturation, and a higher degree of humidity; but beyond this nothing can be infe at, and before any further deduction can be made, rect determinations of the dew point at various h will be necessary.

The places whose state seems to be nearest to that of England ar d Darjiling. Landaur at the same elevation as the same annual temperature of the dew poi unt of water present in the air is nearly the s the amount of water necessary for saturation is s as large as in England, the air is less humid. At all other places the dew point is a great deal higher than in England, and the amount of water actually present as well as that necessary for saturation is greater, so that the air is throughout the whole year, and more especially in the cold weather, much less humid. At certain places, Belgaum, Sattara, Mahableshwar, Dapuli, Bombay, Thayatmyo, Calcutta, and the country thence to Banaras, the air is only in the summer months more humid than in England.

III.—ON THE ADDITIONAL TABLES.

ATMOSPHERIC PRESSURE.

THE daily variation of pressure in India is extremely regular; the minima occur at about 4 A.M. and 5 P.M., the maxima at about 10 A.M. and 11 P.M., or, roughly speaking, at about one or two hours before sunrise, noon, sunset, and midnight; the morning maximum being greater than the evening one, and the evening minimum lower than the morning one. The difference between the mean daily readings seldom exceeds ·2 inches, and the whole daily range is generally less than ·1 inch. The mean change of pressure from year to year is generally very small, and the change from month to month is very constant in different years, the maximum being in January and the minimum in June, the pressure increasing and decreasing regularly throughout the year, the difference being ·26 in the Presidencies of Madras and Bombay, and ·44 in that of Bengal generally. Locally, the distribution of pressure, from the account of Mr. Blanford, is as follows:—

Beginning with October, the month in which the south-west mansun terminates, the pressure is nearly uniform over Burmah, Bengal, Central, Northern, and Eastern India: in November, the pressure rises rapidly over the whole of this area, but more especially in two distinct areas, one being the elevated tract lying south of the Ganges, including Bandalkand, Chota Nagpur, and a part of Nagpur, up to Banaras on the north and down to Cuttack on the south; the others being an area in the Upper Panjab coinciding with the locus of lowest mean winter temperature. The intermediate Gangetic plains on the Gangetic delta, the Malwa plateau, and the flats of Southern Orissa, fall outside both of these areas. In December the general pressure is at its annual maximum, and in January it is nearly as high, all over India, but the pressure is less at Bombay and on the west coast than in Eastern India. It is probable that the fall of pressure with the approach of the hot weather is less rapid in the Panjab than in the Central Provinces and Bengal. In March, April, and May the maximum pressure is about Nagpur, and in the hill country about Hazaribagh it is lower than either on the delta and coast to the east and south-east, or in the Upper Provinces to the west and north-west. In June, the setting in of the south-west mansun is accompanied by a sudden fall of pressure; greater, however, in the Panjab than in the Nagpur region, so that the locus of minimum pressure is probably transferred to the former. In

A table of pressu
gical tables, which

First, as regards
mean daily range of
elevations differs litl
same latitude. The
place do not, on the w

The following are th
the mean monthly tem
of latitude :—

October	−0°·2	
November	−0°·5	
December	−1°·1	
January	−1°·0	Ap
February	−0°·8	
March	−0°·5	

the greatest differences b
30° in amount for the extr
regard to latitude. At mode
ture of the air at a place
which allow

regularly with the increase of elevation above mean sea level up to 9,000 feet, according to the following table:—

Height.	Decrease.	Height.	Decrease.
1000	¾° to 4½°	6000	18° to 27½°
2000	3° to 8¾°	7000	21° to 36°
3000	6½° to 13¼°	8000	23° to 44°
4000	11¾° to 18°	9000	26° to 53°
5000	15° to 23°		

the amounts given being maxima and minima in the year. There is also a regular monthly increase or decrease of high day temperature, due to an increase of one degree of latitude, thus :—

For November	−0°·3	For April	+ 0°·2
„ December	−1°·1	„ May	+ 0°·3
„ January	−0°·7	„ June	+ 0°·4
„ February	−0°·7	„ July	+ 0°·4
„ March	−0°·3	„ August	+ 0°·1
		„ September	+ 0°·2
		„ October	+ 0°·1

The effect of longitude is inappreciable from June to August and for other months, westward stations have a higher day temperature than eastward by a difference of about half a degree for each degree of longitude.

Thirdly. As regards low temperature at night. The effect of latitude on low night temperature is almost inappreciable from May to September; but from November to March the effect is about one degree of temperature for each degree, and in April and October the effect is about half that; the northern stations being colder. The effect of an increase of one degree of east longitude is greatest in places having less than fifteen degrees of latitude; it amounts to a decrease of more than one degree and a half for each degree of greater east longitude in January and February, to a little less than that in March, to three-quarters of a degree in April, and to one quarter of a degree in May After May a change takes place, and from June to September those places with greater east longitude are from a quarter to half a degree warmer for each degree of longitude. The following table gives the decrease of night temperature due to increase of elevation up to 9,000 feet:

Height.	Decrease.	Height.	Decrease.
1000	½° to 3⅛°	6000	13° to 22¼°
2000	3½° to 6½°	ooo	15° to 29°
3000	5¾° to 1	o	17° to 35°
4000	8½° to	o	19° to 41°
5000	10¾° to 1		

the amounts given being xima and minima in the year.

Some statistics of ture of the air, of the temperature of solar k bulb thermometer in vacuo, and of grass ra ous places in India, will be found *in the additional*

WIND AND SERENITY.

The phenomena of the mansuns and general winds of India being better studied from the charts of the large works on physical geography than from any brief account that the limits of this book would allow, it will be unnecessary here to enter into the subject. With regard to local observation of wind in India, comparatively little has been yet done. Mr. Chambers' "Winds of Bombay" gives some valuable information for the year 1867 in a novel form; and the two accompanying tables, taken from the report of Mr. Blanford for 1873, comprise everything else that is of much value. A table of serenity for a few places is also given.

In conclusion, the Meteorological Statistics of India are still too incomplete and irregular to lead to any very important scientific result—in fact, they do not yet arrive at the sufficiency required by the engineer; nevertheless, a judicious use of such data as we possess may, it is hoped, prevent the recurrence of such difficulties as have so frequently occurred from totally ignoring them.

TABLE

(Of Guyot, arranged by Blanford)

FOR FINDING THE RELATIVE HUMIDITY OF THE AIR,

FROM THE READINGS OF WET AND DRY BULB THERMOMETERS,
SATURATION BEING 100.

FOR THE USE OF OBSERVERS.

Relative Humidity Table.

t – t′, or difference of Wet and Dry Bulb Thermometers.
t below the Freezing Point, the Bulb covered with a Film of Ice.

t Fahrenheit.	0·0 Humidity	0·5 Humidity	1·0 Humidity	1·5 Humidity	2·0 Humidity	2·5 Humidity	3·0 Humidity	3·5 Humidity	4·0 Humidity	4·5 Humidity.	5·0 Humidity.	5·5 Humidity.	6·0 Humidity	6·5 Humidity	7·0 Humidity	7·5 Humidity	8·0 Humidity	8·5 Humidity	9·0 Humidity	9·5 Humidity.	10·0 Humidity	10·5 Humidity.	11·0 Humidity	11·5 Humidity
0	100	85	71	57	43	31	19	7																
1	100	85	71	58	46	34	22	11																
2	100	86	73	59	49	36	25	14																
3	100	86	74	61	50	38	28	17																
4	100	87	75	63	52	41	30	20	11															
5	100	87	76	64	53	43	33	23	14															
6	100	88	76	66	55	45	35	26	17	9														
7	100	88	77	67	56	47	37	28	20	12														
8	100	89	78	68	58	48	39	31	22	15														
9	100	89	79	69	59	50	41	33	25	17	10													
10	100	89	79	70	61	52	43	35	28	20	13													
11	100	90	80	71	62	53	45	37	30	23	16													
12	100	90	81	72	63	55	47	40	32	25	19	12	6											
13	100	90	81	73	64	56	49	42	35	28	21	15	9											
14	100	91	82	73	65	58	50	43	37	30	24	18	12	7										
15	100	91	82	74	67	59	52	45	39	32	26	21	15	10	5									
16	100	91	83	75	68	61	54	47	41	35	29	23	18	13	8									
17	100	91	84	76	69	62	55	49	43	37	31	26	21	16	11	6								
18	100	92	84	77	[illegible]	63	57	50	44	39	33	28	23	18	14	9	5							
19	100	92	85	78	71	64	58	52	46	41	35	30	25	21	16	12	8	4						
20	100	92	85	78	72	65	59	54	48	43	37	32	28	23	19	15	11	7	3					
21	100	92	86	79	73	66	61	55	50	44	39	34	30	25	21	17	13	9	6	2				
22	100	93	86	80	74	68	62	57	51	46	41	36	32	28	23	19	16	12	8	5	2			
23	100	93	86	80	74	69	64	58	52	48	43	38	34	30	26	22	18	14	11	8	4			
24	100	93	87	81	75	69	64	59	54	49	44	40	36	32	28	24	20	17	13	10	7	4		
25	100	93	87	81	76	70	65	60	55	51	46	42	38	34	30	26	22	19	16	13	10	7	4	
26	100	94	88	82	76	71	66	61	56	52	48	43	39	36	32	28	25	21	18	15	12	9	6	4
27	100	94	88	82	77	72	67	62	58	53	49	45	41	37	34	30	27	23	20	17	14	11	8	6
28	100	94	88	83	78	73	68	63	59	55	51	47	43	39	35	32	29	25	22	19	16	14	11	8
29	100	94	89	83	78	74	69	64	60	56	52	48	44	41	37	34	31	27	24	21	19	16	13	11
30	100	94	89	84	79	74	70	65	61	57	53	49	46	42	39	35	32	29	26	23	21	18	15	13
31	100	94	89	84	80	75	71	66	62	58	55	51	47	44	40	37	34	31	28	25	22	20	17	15

Relative Humidity Table.

Wet Bulb Thermometer, t Fahrenheit	t - t′, or difference of Wet and Dry Bulb Thermometers.																	
	0·0	0·5	1·0	1·5	2·0	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0	6·5	7·0	7·5	8·0	[illegible]
	Humidity	Humidity	Humidity	Humidity.	Humidity	[illegible]	[illegible]	[illegible]	[illegible]	Humidity	Humidity	Humidity	Humidity.	Humidity.	Humidity	Humidity	Humidity	Humidity.
32	100	94	89	84	79	74	[illegible]	[illegible]	[illegible]	56	52	48	45	41	37	34	31	[illegible]
33	100	94	89	84	79	75	[illegible]	[illegible]	[illegible]	57	53	50	46	42	39	36	33	[illegible]
34	100	94	89	84	80	77	[illegible]	[illegible]	[illegible]	58	54	51	47	44	40	37	34	31
35	100	94	90	85	80	[illegible]	[illegible]	[illegible]	[illegible]	59	55	52	48	45	42	38	35	[illegible]
36	100	95	90	85	81	[illegible]	[illegible]	[illegible]	[illegible]	60	56	53	50	46	43	40	37	[illegible]
37	100	95	90	86	81	[illegible]	[illegible]	[illegible]	[illegible]	61	57	54	51	47	44	41	38	[illegible]
38	100	95	90	86	82	[illegible]	[illegible]	[illegible]	[illegible]	62	58	55	52	49	45	42	40	[illegible]
39	100	95	91	86	82	[illegible]	[illegible]	[illegible]	[illegible]	63	59	56	53	50	47	44	41	[illegible]
40	100	95	91	86	82	78	[illegible]	[illegible]	[illegible]	64	60	57	54	51	48	45	42	39
41	100	95	91	87	83	79	75	72	68	65	61	58	55	52	49	46	43	41
42	100	95	91	87	83	79	76	73	69	65	62	59	56	53	50	47	45	42
43	100	95	91	87	84	80	76	73	69	66	63	60	57	54	51	48	46	43
44	100	95	92	88	84	80	77	73	70	67	63	61	58	55	52	49	47	44
45	100	96	92	88	84	81	77	74	71	67	64	61	59	56	53	50	48	45
46	100	96	92	88	85	81	78	74	71	68	65	62	59	57	54	51	49	46
47	100	96	92	88	85	81	78	75	72	68	66	63	60	57	55	52	50	47
48	100	96	92	89	85	82	79	75	72	69	66	64	61	58	56	53	51	48
49	100	96	92	89	85	82	79	76	73	70	67	64	62	59	56	54	52	49
50	100	96	92	89	86	82	79	76	73	70	68	65	62	60	57	55	53	50
51	100	96	93	89	86	83	80	77	74	71	68	66	63	60	58	56	53	51
52	100	96	93	89	86	83	80	77	74	71	69	66	64	61	59	56	54	52
53	100	96	93	90	86	83	80	78	75	72	69	67	64	62	59	57	55	53
54	100	96	93	90	87	84	81	78	75	72	70	67	65	62	60	58	56	53
55	100	96	93	90	87	84	81	78	76	73	70	68	65	63	[illegible]	59	56	54
56	100	96	93	90	87	84	81	79	76	73	71	68	66	64	61	59	57	55
57	100	96	93	90	87	85	82	79	76	74	71	69	67	64	62	60	58	56
58	100	96	93	90	88	85	82	79	77	74	72	69	67	65	63	60	58	56
59	100	96	94	91	88	85	82	80	77	75	72	70	68	65	63	61	59	57
60	100	97	94	91	88	85	83	80	78	75	73	70	68	66	64	62	60	58
61	100	97	94	91	88	85	83	80	78	75	73	71	69	66	64	62	60	59
62	100	97	94	91	88	86	83	81	78	76	74	71	69	67	65	63	61	59
63	100	97	94	91	89	86	83	81	79	76	74	72	70	67	65	63	61	59
64	100	97	94	91	89	86	84	81	79	77	74	72	70	68	66	64	62	60
65	100	97	94	91	89	86	84	81	79	77	75	72	70	68	66	64	62	61
66	100	97	94	92	89	87	84	82	79	77	75	73	71	69	67	65	63	61
67	100	97		92	89	87	84	82	80	78	75	73	71	69	67	65	63	62

Relative Humidity Table.

Wet Bulb Thermometer t Fahrenheit	t—t′, or difference of Wet and Dry Bulb Thermometers.																	
	0·0	0·5	1·0	1·5	2·0	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0	6·5	7·0	7·5	8·0	8·5
	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity.	Humidity	Humidity	Humidity	Humidity	Humidity
68	100	97	94	92	89	87	85	82	80	78	76	74	72	70	68	66	64	62
69	100	97	94	92	89	87	85	82	80	78	76	74	72	70	68	66	64	63
70	100	97	94	92	90	87	85	83	81	78	76	74	72	70	68	67	65	63
71	100	97	95	92	90	87	85	83	81	79	77	75	73	71	69	67	65	64
72	100	97	95	92	90	88	85	83	81	79	77	75	73	71	69	67	66	64
73	100	97	95	92	90	88	86	83	81	79	77	75	73	71	70	68	66	64
74	100	97	95	92	90	88	86	84	81	79	77	76	74	72	70	68	66	65
75	100	97	95	92	90	88	86	84	82	80	78	76	74	72	70	68	67	66
76	100	97	95	93	90	88	86	84	82	80	78	76	74	72	71	69	67	66
77	100	97	95	93	90	88	86	84	82	80	78	76	75	73	71	69	68	66
78	100	97	95	93	91	88	[illegible]	84	82	80	78	77	75	73	71	70	68	66
79	100	97	95	93	91	89	87	85	83	81	79	77	75	73	72	70	68	67
80	100	97	95	93	91	89	87	85	83	81	79	77	75	74	72	70	69	67
81	100	97	95	93	91	89	87	85	83	81	79	77	76	74	72	71	69	67
82	100	97	95	93	91	89	87	85	83	81	79	78	76	74	73	71	69	68
83	100	97	95	93	91	89	87	85	83	81	80	78	76	74	73	71	70	[illegible]
84	100	97	95	93	91	89	[illegible]	85	83	82	[illegible]	78	76	75	73	71	70	68
85	100	97	95	93	91	89	87	85	84	82	80	78	77	75	73	72	70	69
86	100	97	96	93	91	[illegible]	87	86	84	82	80	79	77	75	74	72	70	69
87	100	97	95	93	91	90	88	86	84	82	[illegible]	79	77	75	74	72	71	69
88	100	97	95	93	92	90	88	86	[illegible]	82	[illegible]	79	77	[illegible]	74	73	71	70
89	100	97	95	94	92	90	88	86	84	83	81	79	[illegible]	76	74	73	71	70
90	100	98	96	94	92	90	[illegible]	[illegible]	84	83	81	79	78	76	75	73	72	70
91	100	98	96	94	92	90	88	86	85	83	81	80	78	76	75	73	72	70
92	[illegible]	98	96	94	92	90	88	86	85	83	81	80	78	77	75	74	72	71
93	100	98	96	94	92	[illegible]	88	87	85	[illegible]	82	80	78	77	75	74	72	71
94	100	98	96	94	92	90	88	87	85	83	82	80	79	77	75	74	73	71
95	100	98	[illegible]	94	92	90	88	87	85	83	82	80	79	77	76	74	[illegible]	71
96	100	98	96	94	92	90	89	87	85	[illegible]	82	[illegible]	79	77	76	74	73	72
97	100	[illegible]	96	94	[illegible]	[illegible]	89	87	85	84	82	81	79	78	76	75	73	72
98	100	98	96	94	92	90	89	87	85	84	[illegible]	81	79	78	76	75	73	72
99	100	98	96	94	92	91	89	87	86	84	82	81	79	78	76	75	74	72
100	100	98	96	94	92	91	89	87	86	[illegible]	83	81	80	78	77	[illegible]	74	[illegible]
101	100	98	96	94	92	91	89	87	86	84	83	81	80	78	77	75	74	73
102	100	98	96	94	92	91	89	88	86	84	83	81	80	78	77	76	74	73
103	100	98	96	94	93	91	89	88	86	84	83	81	80	79	77	76	74	73
104	100	98	96	94	93	[illegible]	89	88	86	85	83	82	80	79	77	76	75	73

Relative Humidity Table.

t° Fahrenheit.	t – t′, or difference of Wet and Dry Bulb Thermometers.—Fahrenheit.																	
	9.0	9.5	10.0	10.5	11.0	11.5	12.0	12.5	13.0	13.5	14	14.5	15.0	15.5	16.0	16.5	17.0	17.5
	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity.	Humidity	Humidity	Humidity.	Humidity	Humidity	Humidity	Humidity	Humidity
68	60	59	57	55	54	52	51	49	48	46	45	44	42	41	40	39	37	36
69	61	59	58	56	54	53	51	50	48	47	46	44	43	42	41	39	38	37
70	61	60	58	56	55	53	52	50	49	48	46	45	44	43	41	40	39	38
71	62	60	59	57	55	54	52	51	50	48	47	46	44	43	42	41	40	39
72	62	61	59	58	56	54	53	52	50	49	48	46	45	44	43	42	40	39
73	63	61	60	58	56	55	54	52	51	49	48	47	46	44	43	42	41	40
74	63	62	60	58	57	56	54	53	51	50	49	47	46	45	44	43	41	40
75	64	62	60	59	57	56	55	53	52	51	49	48	47	46	44	43	42	41
76	64	63	61	59	58	56	55	54	52	51	50	[illegible]	47	46	45	44	43	42
77	64	63	61	60	58	57	56	54	53	52	50	49	48	47	45	44	43	42
78	65	63	62	60	59	57	56	55	53	52	51	50	48	47	46	45	44	43
79	65	64	62	61	59	58	56	55	54	53	51	50	49	48	47	45	44	43
80	66	64	63	61	60	58	57	56	54	53	52	51	49	48	47	46	45	44
81	66	64	63	62	60	59	57	56	55	53	52	51	50	49	48	46	45	44
82	66	65	63	62	[illegible]	59	58	56	55	54	53	51	50	49	48	47	46	45
83	67	65	64	62	61	59	58	57	56	54	53	52	51	50	48	47	46	45
84	67	65	64	63	61	60	59	57	56	55	54	52	51	50	[illegible]	48	47	46
85	67	66	64	63	62	[illegible]	59	58	56	55	54	53	52	50	[illegible]	48	47	46
86	68	66	65	63	62	61	59	58	57	56	54	53	52	51	50	49	48	47
87	68	66	65	64	62	61	60	58	57	56	55	54	52	51	50	49	48	47
88	68	67	65	64	63	61	60	59	58	56	55	54	53	52	51	50	49	48
89	68	[illegible]	66	64	63	62	60	59	58	57	56	54	53	52	51	50	[illegible]	48
90	69	67	66	65	63	62	61	59	58	57	56	55	54	53	51	50	49	48
91	69	68	66	65	64	62	61	60	59	57	56	55	54	53	52	51	50	49
92	69	68	67	65	64	63	61	60	59	58	57	55	54	53	52	51	50	[illegible]
93	69	68	67	65	64	63	62	60	59	[illegible]	57	56	55	54	53	52	51	50
94	70	68	67	66	64	63	62	61	60	58	57	56	55	54	53	52	51	50
95	70	69	67	66	65	64	62	61	[illegible]	59	58	57	55	54	53	52	51	50
96	70	69	68	[illegible]	65	64	63	61	60	59	58	57	56	55	54	53	52	51
97	70	69	68	67	65	64	63	62	61	59	58	57	56	55	54	53	52	51
98	71	69	68	67	66	64	63	62	61	60	59	57	56	55	54	53	52	51
99	71	70	68	67	66	65	63	[illegible]	61	60	59	58	[illegible]	56	55	54	53	52
100	71	70	69	67	66	65	64	63	62	60	59	58	57	56	55	54	53	52
101	71	70	69	68	66	65	64	63	62	61	59	58	57	56	55	54	53	52
102	72	70	69	68	67	66	64	63	62	61	60	59	58	57	56	55	54	[illegible]
103	72	71	69	68	67	66	64	63	62	61	60	59	58	57	56	55	54	53
104	72	71	69	68	67	66	65	64	63	61	60	59	58	57	56	55	54	53

Wet Bulb	Humidity	Humidity	Humidity	Humidity			
32							
33							
34							
35							
36							
37							
38							
39							
40	2						
41	4	2	1				
42	6	4	2	2			
43	7	6	3	3	2		
44	9	7	5	5	4	1	
45	10	9	6	7	5	4	
46	12	11	9	8	7	6	
47	13	12	11	10	9	7	
48	15	13	12	11	10	9	
49	16	15	14	12	11	10	
50	18	16	15	14	13	12	
51	19	18	16	15	14	13	
52	20	19	18	16	15	14	
53	21	20	19	18	16	15	1
54	22	21	20	19	18	17	1
55	24	22	21	20	19		
56	25						

Relative Humidity Table.

t—t′, or difference of Wet and Dry Bulb Thermometers.—Fahrenheit.																	
18°·0	18°·5	19°·0	19°·5	20°·0	20°·5	21°·0	21°·5	22°·0	22°·5	23°·0	23°·5	24°·0	24°·5	25°·0	25°·5	26°·0	26°·5
Humidity.	Humidity	Humidity	Humidity	Humidity	Humidity.	Humidity.	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity	Humidity.	Humidity
35	34	33	32	31	30	29	28	27	26	25	25	24	23	22	21	21	20
36	35	34	33	32	31	30	29	28	27	26	25	24	24	23	22	21	21
37	36	35	33	32	31	31	30	29	28	27	26	25	24	24	23	22	21
37	36	35	34	33	32	31	30	29	29	28	27	26	25	24	24	23	22
38	37	36	35	34	33	32	31	30	29	28	28	27	26	25	24	24	23
39	38	37	36	35	34	33	32	31	30	29	28	27	27	26	25	24	24
39	38	37	36	35	34	33	32	31	31	30	29	28	27	27	26	25	24
40	39	38	37	36	35	34	33	32	31	30	30	29	[illegible]	27	26	26	25
40	39	38	37	36	35	35	34	33	32	31	30	29	29	28	27	26	26
41	40	39	38	37	36	35	34	33	33	32	31	30	29	28	28	27	26
42	41	40	39	38	37	36	35	34	33	32	31	31	30	29	28	28	27
42	41	40	39	38	37	36	35	35	34	33	32	31	30	30	29	28	27
43	42	41	40	39	38	37	36	35	34	33	33	32	31	30	29	29	28
43	42	41	40	39	38	37	37	36	35	34	33	32	32	31	30	29	29
44	43	42	41	40	39	38	37	36	35	35	34	33	32	31	31	30	29
44	43	42	41	40	39	39	38	37	36	35	34	33	33	32	31	30	30
45	44	43	42	41	40	39	38	37	36	36	35	34	[illegible]	32	32	31	30
45	44	43	42	41	40	40	39	38	37	36	35	35	34	33	32	32	31
46	45	44	43	42	41	[illegible]	39	38	37	37	36	35	34	33	33	32	31
46	45	44	43	42	41	[illegible]	40	39	38	37	36	36	35	34	33	33	32
47	46	45	44	43	42	41	40	39	38	38	37	36	35	34	34	33	32
47	46	45	44	43	42	41	41	40	39	38	37	36	36	35	34	33	33
47	46	45	45	44	43	42	41	40	39	38	38	37	36	35	35	34	33
48	47	46	45	44	43	42	41	41	40	39	38	37	37	[illegible]	35	34	34
48	47	46	45	44	44	43	42	41	40	39	39	38	37	36	36	35	34
49	48	47	46	45	44	43	42	41	41	40	39	38	37	37	36	35	35
49	48	47	46	46	44	43	43	42	41	40	39	39	38	37	36	[illegible]	33
49	48	47	47	46	45	44	43	42	41	41	40	39	38	37	37	36	35
50	49	48	47	46	45	44	43	43	42	41	40	39	39	[illegible]	37	37	36
50	49	49	47	46	46	45	44	43	42	41	41	40	39	38	38	37	36
50	49	49	48	47	46	45	44	43	43	42	41	40	39	39	38	37	37
51	50	49	48	47	46	45	45	44	43	42	41	41	40	39	38	39	37
51	50	49	48	47	47	[illegible]	45	44	43	43	42	41	40	[illegible]	39	38	37
51	50	50	49	48	47	46	45	44	44	43	42	41	41	40	39	38	38
52	51	50	49	48	47	46	46	45	44	43	42	42	41	40	40	39	38
52	51	50	49	48	48	47	46	45	44	44	43	42	41	41	40	39	38
52	51	51	50	49	48	47	46	45	45	44	43	42	42	41	40	40	39

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